STATE OF CALIFORNIA
HIGHWAY TRANSPORTATION AGENCY
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

EXPERIMENTAL SAND DRAIN FILL AT NAPA RIVER

By
William G. Weber Jr.
Senior Materials & Research Engineer

Presented at
45 th Annual Highway Research Board Meeting
Washington D.C.
January 17 to 21, 1966

66-15

EXPERIMENTAL SAND DRAIN FILL AT NAPA RIVER

By

William G. Weber, Jr.*

ABSTRACT

The use of sand drains to accelerate the consolidation of soft fine grain soils has been widely used in California. Difficulty has been encountered in predicting the action of sand drains during the design of these projects. The experimental fill at Napa River was constructed to obtain information to better understand the action of sand drains. It was found that sand drains did not accelerate the consolidation of the soft foundation soil as rapidly as the theory would indicate. It appears that the method of placing the sand drains with a closed mandrel was one of the main causes of this reduction in the expected rate of consolidation.

*Senior Materials and Research Engineer, Materials and Research Department, Division of Highways, Sacramento, California.

EXPERIMENTAL SAND DRAIN FILL AT NAPA RIVER

by

William G. Weber, Jr.*

The use of sand drains to stabilize soft foundation soils has been widely used by the California Division of Highways and other organizations since 1934. There have been many successful projects and some others that did not perform as the theory indicated. In 1957 a review of all previous sand drain projects in the Division of Highways was conducted to determine why some of the projects were not performing as expected. It was noted that the questionable projects were principally in the soft silty clays of recent geological origin. The decision was then made to conduct an extensive test program at a sand drain installation constructed in an area where a soft silty clay foundation soil existed. The rate of consolidation and strength increase of the soft foundation soil could then be studies and compared to the theoretical solutions. It was desired to construct a step type fill where sand drain and nonsand drain areas could be compared with the same fill loading. Also where the fill height was sufficient that the stability

*Senior Materials and Research Engineer, Materials and Research Department, Division of Highways, Sacramento, California

would be questionable. It was hoped to thus decrease some of the uncertainties in the design of sand drain installations in soft silty clay foundation soils.

It was desired to conduct this test section to determine why the sand drains have not accelerated the consolidation of the soft foundation soil as anticipated by theory in all of the projects. A comparison of areas with and without sand drains at constant height of fill was desired to determine the amount that sand drains accelerated the consolidation of the foundation soil. The affect of sand drains upon the stability of the fill was also to be studied. The west approach to the Napa River bridge on Route 37 in Solano County provided the desired location for a test section.

Site Description

The foundation soil at the west approach to the Napa River consisted of "soft bay mud" to a depth of 60 to 70 feet and was reasonably homogeneous, with a firm silty clay underlying the soft bay mud. The natural water contents average above 90 percent and the in-place strength varied from about 100 lbs. per square foot at a depth of ten feet to near 500 lbs. per square foot at a depth of 40 feet. The average soil properties are shown in Table No. 1. Without any special treatment this soil will support fills of about six feet in height. With the use of struts it is possible to build somewhat higher fills.

The height of fill to produce the planned profile grade at the west approach to the Napa River Bridge varied from less than five feet to over 50 feet above the mud flats, depending upon where the structure ended. Construction of the fill would end, and the structure begin, when no special stabilization was used where the profile grade was about five feet above the mud flats. The cost of sand drain treatment and fill construction was estimated at about $120,000.00 per station less than a structure.

The new alignment was adjacent to the existing roadway, thus avoiding interference with existing traffic. The west approach fill could be constructed prior to the structure under a separate contract, allowing sufficient time to construct and observe the fill. (See Figure No. 1).

The soft bay mud was very impervious, having a permeability of 10^{-5} feet per hour. Theoretical calculations[1] indicated, (1) that sand drains on an 8-foot spacing would consolidate the soil at a rate that would increase the strength sufficiently during construction to allow a rate of loading of one and one-half foot of fill per week with waiting periods required for the higher fill heights. In previous sand drain construction[2] on similar soils theoretical solutions indicated a spacing of ten feet with a rate of loading of three feet of fill per week was required. This difference between Napa River and the previous sand drain projects was that the upper ten feet of the foundation soil at Napa River had a very low shearing strength; (2) with fills above 15 feet in height berms 100 feet wide and eight feet high were required on the left side and the existing roadway acted as a berm on the right side of the fill. Sand drains were installed under the main fill only. (See Figure No. 1 and 2).

Construction

In February, 1960, the construction of 1,200 feet of fill as a bridge approach on the west side of the Napa River was begun. This was to be an experimental project and was to be a step-type surcharge fill with struts. After the designed height was reached, a waiting period would begin to allow observation of the fill.

Due to the extremely swampy nature of the foundation soil, a working platform was constructed over the tules and swamp grass. With the exception of a few large timbers, no clearing was performed. Care was taken to keep the working platform as thin as possible, and it generally was about five feet in thickness. An occasional mud wave developed and when this occurred the mud wave was trapped by placing fill around it and the mud wave covered.

Approximately 2,500 sand drains were then driven, varying in depth from 42 to 72 feet, using an 18-inch hollow mandrel with a hinged bottom. After the mandrel had been driven and the sand placed inside compressed air was applied at the top forcing the sand out of the bottom as the steel mandrel was withdrawn. When the drains were completed, a 2- to 3-foot blanket of filter material was placed over the working table. The grading of the sand used in the sand drains and the filter material used in the blanket are shown in Table No. 2. To aid drainage of water from the filter blanket an 8-inch perforated metal pipe (PMP) was placed along the centerline. Construction of the fill was then started at a rate of one and one-half feet per week. After the main fill reached a height of about 17 feet cracks appeared, with the right side tending to move excessively (see Figure No. 2).

The placing of embankment on the main fill was discontinued and a strut constructed on the right side. Traffic was temporarily detoured to the right until additional fill could be placed on top of the existing pavement. After this fill was brought to an elevation of 14 feet, traffic was rerouted to the original location with the right strut reinforced as shown on Figures No. 1 and 2. As there were some indications that the left side was moving, the struts were widened to 160 feet on the left and their height near the main fill increased. See Figures No. 1 and 2.

On September 27, with the main fill at an elevation of 22 feet and about 22 feet thickness of fill, the left side of the fill showed excessive movement. Numerous cracks from two to seven feet in width appeared. The placing of embankment on the main fill was discontinued and a waiting period begun. This contract was closed on January 15, 1961.

Between March and June of 1962, the elevation of the main fill from Station 394+00 to Station 396+80 was increased to elevation 24 feet. The fill was at about elevation 17 feet at the start of this loading. The fill was placed at a rate of one foot per week, and was obtained from the existing fill between Station 396+50 and 400+00. The structure will now begin at Station 396+00.

Failures

The movement when the main fill was at elevation +17 was minor in nature. The piezometers in the soft foundation soil to the right of the existing roadway showed an increase in pressure of 0.2 ton per square foot in about one to two hours' time. The slope indicator to the right of the roadway indicated that the upper 15 to 20 feet of soft mud below the roadway had moved slightly outward. The heave line on the existing roadway indicated an acceleration in the movement of the roadway surface to the right. Minor cracking occurred near the centerline of the fill with cracks opening less than one inch. Loading of the main fill was immediately stopped. Readings the next day indicated that the movement of the roadway was continuing so additional berm was then placed on the right side of the fill.

A cross-section of the fill is shown in Figure No. 3 along with the minimum failure arc. Stability analysis indicated that no significant increase in strength of the underlying foundation soil had occurred. Two minimum failure arcs with factors of safety just below unity were located and indicated that heave of the roadway or existing fill was occurring. This was in agreement with the observations that no visible movement of the mud surface to the right of the existing fill occurred, and that the roadway fill was rising.

The movement on the right raised questions as to the ability of the berm on the left to support the main fill. To increase the stability of the main fill the primary berm on the left was widened 60 feet and the existing 100-foot berm increased in height to elevation 14 feet. To provide drainage of the filter blanket 8-inch PMP were installed through the widened berm. The increase in the widths of the struts are shown in Figure No. 2. On September 8, 1960, the loading of the main fill was resumed.

On September 27, 1960, a major movement occurred on the left side as shown in Figure Nos. 1 and 4. The piezometers at the toe of the main fill had been indicating a slow steady rise in excess hydrostatic pressure prior to the failure. The heave stakes had indicated a small steady outward movement, and the slope indicators showed that a minor flow of the soft bay mud was occurring at about elevation -10 feet. This movement occurred very rapidly with about a 4-foot drop in the left side of the fill.

The stability analysis of this movement are shown in Figure No. 4. The shear circle analysis indicated a factor of safety of unity existed. Using the stability analysis for plastic type movements developed from analysis of the failures at Candlestick Cove[3], the results indicated a factor of safety of 0.7. The rapid development of a large mud wave and the general appearance of the area indicated that this plastic type of failure had occurred. There was no indication that a significant increase in strength of the soft foundation soil had occurred.

This shallow shear failure through the upper portions of the soft foundation soil was not expected. It had been expected that the upper portions of the soft foundation soil would consolidate rapidly, increasing the shearing strength of the soil immediately below the berm. This would have resulted in a deeper failure surface with an increased length of the failure plane and in a higher strength soil. This failure surface in the upper portion of the soft foundation soil indicated very wide berms would be required to increase the length of the failure surface so that a stable fill could be constructed. This wide berm would have been uneconomical to construct. An alternative to such a wide berm would be the stabilization of the foundation soil under the berm by means of sand drains.

Installations

A total of more than 200 installations such as settlement platforms, piezometers, borings, and other instrumentations were made at the west approach to the Napa River Bridge.

A total of 19 sample borings were made prior to construction of the fill. During and after construction 40 sample borings have been made. These borings were primarily to determine the strength and moisture content of the foundation soil. The sample borings were supplemented with 13 vane borings[4].

A total of 25 settlement platforms were installed on the surface of the working table. Four settlement platforms were also installed at the bottom of the soft foundation soil. The settlement platforms were to measure the vertical movement of the surface upon which they were placed.

A total of 106 piezometers were placed at various locations to measure the excess hydrostatic pressures developed. Seven piezometers were placed in the filter material to measure the excess hydrostatic pressure being developed on the filter system. Forty piezometers were installed for stability studies at the edges of the fill. Fifteen piezometers were installed in the soft foundation soil under the berms to study their rates of consolidation. Forty-four piezometers were placed in the soft foundation soils below the main fill to measure the rate of consolidation in the sand drain and nonsand drain areas. All of the piezometers were of the nonmetalic type similar to those developed for use at Logan Airport[5].

The movement of the soft foundation soil was measured using nine slope indicators[6]. The movement of the fill surfaces were measured by the use of six heave stake lines for a total of 4,700 linear feet of heave stake lines.

Water tables were determined by means of 12 water table tubes in the fill area and in the foundation soil outside the fill area. These water table tubes consisted of perforated 3/4-inch pipe place in an augered 3-inch diameter hole and back-filled with sand.

The changes in the moisture and density of the soft foundation soil below the fill was measured by a nuclear subsurface gage and eight nuclear access tubes. The access tubes consisted of steel tubing with 1/16-inch wall thickness. The Nuclear Chicago subsurface nuclear probes were used to measure the changes in soil moisture and density[7].

Settlements

The settlements of the main fill along centerline are shown in Figure No. 5. The settlements in the sand drain areas were about twice that in the nonsand drain areas during construction. After completion of construction, the sand drain areas have continued to settle at this same rate. The settlement in the sand drain areas are, in 1965, approaching the estimated ultimate settlement under the three heights of fill involved.

Typical time settlement plots are shown in Figures No. 6, 7 and 8. The estimated theoretical rate of settlement for the non-sand drain areas were calculated using Terzaghi's equations for double drainage[8]. The theoretical rate of settlement for the sand drain areas are calculated using the equations developed by R. A. Barron with no correction for well resistance, smear zone and soil disturbance[1].

The settlements in the nonsand drain areas have occurred about twice as fast as was estimated. For example, it was estimated that about 750 days would be required for two feet of settlement to occur under 15 feet of fill, whereas about 450 days were actually required. (See Figure No. 7). There is a tendency now, in 1965, for the settlement in the nonsand drain areas to indicate a decreased rate on a logrithmic plot.

There are three possible reasons for the nonsand drain areas showing an accelerated rate of settlement: (1) the adjacent sand drain areas provided some horizontal drainage that would accelerate the consolidation in the nonsand drain areas; (2) the increased rate of settlement of the adjacent sand drain areas produced a drag down on the fill in the nonsand drain areas increasing the effective loading; (3) settlement due to plastic flow of the soft foundation soil. Considering these factors and the normal uncertainties in the calculation of the rate of settlement reasonable agreement has been obtained between the measured and theoretical rates of settlement.

The settlements in the sand drain areas required about twice the time that was estimated from the application of R. A. Barron's equations[1]. This resulted in a much smaller increase in strength of the foundation soil during construction than had been anticipated. There are several possible reasons for this reduced settlement. The resistance of the flow of water through the filter system would reduce the rate of settlement. This will be discussed in the excess hydrostatic pressure section. The reduction in permeability of the foundation soil due to remolding when the sand drains are driven. By using the piezometers as permeameters[9] the permeability of the soil near the sand drain was measured and appeared to decrease greatly. This will be discussed in detail in the excess hydrostatic pressure section. The peripheral smear produced by the drag of the mandrel could form a highly impervious zone. There was no way to measure this factor directly in this test section. The indications are that when the equations developed by Mr. R. A. Barron are used with soil and installation conditions similar to those at the west approach to Napa River the time for consolidation should be increased by a factor of two to two and one-half.

An attempt was made to evaluate the above factors by using the equation developed by R. L. Schiffman[10]. In his paper various factors affecting the rate of consolidation in sand drain areas are mathematically analyzed. The effect of various horizontal and vertical permeability and coefficients of consolidation can be evaluated. Also the effect of peripheral smear can be studied. The ratio between the horizontal and vertical permeabilities averaged two and the ratio between the horizontal and vertical coefficients of consolidation varied from two to three. Using these values and the measured settlement, the zone of peripheral smear was indicated to be about three feet from the center of the sand drain. This would indicate that placing the sand drains by the closed mandrel method remolded the soil for an area about two feet around the sand drain.

The settlement platforms placed at the bottom of the soft mud indicated that less than one foot of settlement of the underlying stiff muds had occurred, as of 1965. This would indicate that no appreciable error is involved in assuming that the measured settlement of the top of the working table represents the consolidation of the soft foundation soil.

Excess Hydrostatic Pressures

Piezometers were installed for two primary purposes. One was to detect instability of the fill during construction. These piezometers were placed near the toe of the main fill and at the toe of the berm, generally in the upper 20 to 30 feet of the foundation soil. The other purpose was to measure the rate of consolidation of the foundation soil. These were placed near centerline of the main fill and in the center of the struts at about each ten feet of depth of the soft foundation soil in both the sand drain and nonsand drain areas to measure vertical drainage. To measure the horizontal drainage in the sand drain areas piezometers were also placed at elevation -15 in cross-section between two sand drains. Thus, vertical drainage in the sand drain and nonsand drain areas, and also horizontal drainage in the sand drain areas were measured.

The piezometers placed at the toe of the fills for the predication of unstable conditions were partially successful in predicting failures. The shear type movement that occurred on the right caused an increase in excess hydrostatic pressure. (See Figure No. 3). These increases in excess hydrostatic pressure were about 0.2 ton per square foot. The warning given by these piezometers prevented a serious failure from occurring. However, with the plastic type failure that occurred on the left side, the piezometers did not show an excessive rise in excess hydrostatic pressure (See Figure No. 4). As the main fill was loaded, the piezometers at the toe of the main fill indicated a slight rise in pressure as would be expected due to the spreading of the fill load in the foundation soil. These piezometers were being read three times a day; morning, noon, and evening. At the time of the movement of the fill on the left side a rise of about 0.1 ton per square foot or less was noted in the excess hydrostatic pressure. This pressure then returned in a few days to the pressure prior to the movement on the left side of the fill. The plastic flow type of movement thus appears to occur without a major rise in pore pressure prior to the movement. This will limit the value of the piezometers when this type of movement may occur.

Limited piezometers were placed in the filter material. These piezometers indicated that generally about 0.1 to 0.2 tons per square foot excess hydrostatic pressure was required to cause the water to flow through the filter material. As the quantity of flow of water decreased after completion of the fill the pressure rapidly approached zero. The pressure of 0.2 tons per square foot is in reasonable agreement with the calculated pressure assuming that the volume of water passing through the filter material is represented by the settlement of the fill. These piezometers indicate the value of placing PMP in the filter blanket to reduce the drainage path through the filter blanket and the need for a high permeability in the filter blanket. Occasionally a pressure of up to 0.5 ton per square foot was recorded in the sand drains. However, this high pressure occurred only infrequently and did not continue for a long period of time.

Piezometers were placed at various depths in the foundation soil in the nonsand drain areas below the main fill and the berm, to measure the rate of consolidation of the foundation soil without sand drains. A typical set of data is shown in Figure No. 10, which is in the berm area to the left of the main fill. As the piezometers were read every other day during construction only representative data is shown during this period. The excess hydrostatic pressures rose during the loading of the foundation soil and decreased when the load remained constant. The excess hydrostatic pressure reached a maximum of 0.65 tons per square foot under a fill loading of 0.70 tons per square foot. After completion of the fill the pressure has slowly dissipated until in 1965 about 60 percent consolidation has occurred. This is in agreement with the settlement data.

In Figure No. 11 this same data has been plotted with excess hydrostatic pressure as a function of depth at given times. The data indicates that at the time of the failure on the left side no significant consolidation of the foundation soil had occurred, which would indicate no increase in strength occurred at that time under the berms. The piezometers near the top and bottom of the layer started indicating a decrease in excess hydrostatic pressure by 1961, indicating that drainage had been occurring. By 1962 a significant decrease in excess hydrostatic pressure in the center of the layer was recorded. At the present time the excess hydrostatic pressure is decreasing as was indicated by the Terzaghi theory of consolidation. The piezometer near the bottom of the layer indicates that the sandy clay and/or peat layer is providing drainage for the soft clay layer. The above data is similar to the data at four other locations in nonsand drain areas and is in agreement with the settlement data at these locations.

Piezometers were also placed at various depths in the sand drain area, and were within one foot of being equal distance from four sand drains. A typical set of this data is in Figure

No. 12. The excess hydrostatic pressure increased as loading occurred and then decreased as the load was maintained constant. The maximum excess hydrostatic pressure was 1.0 ton per square foot at the end of loading under a load of about 1.4 ton per square foot. In Figure No. 13 this same data has been plotted with excess hydrostatic pressure as a function of depth at given times. The data indicates that at the time of the failure on the left side about 40 percent consolidation had occurred. The settlement curves at the location of these piezometers are shown in Figure No. 8, and the settlement data is in reasonable agreement with the excess hydrostatic data. The piezometers below elevation -35 failed to function properly after loading was completed so that the data is incomplete for this period. these malfunctions occurred for most piezometers at greater depth after loading was completed and the lines appeared to be pinched. That is a wire would not go through the plastic tubing to the porous stone, but would be stopped above the porous stone[5]. Prior to placing the additional load in 1962 the piezometers indicated about 70 percent consolidation had occurred which is in agreement with the settlement data. At the present time the piezometer data indicates that about 90 percent consolidation has occurred whereas the settlement data indicates that the settlement is completed. This minor discrepancy is not considered serious. This data is typical of the data obtained at five other locations in various sand drain areas.

On previous sand drain projects abnormally high excess hydrostatic pressures had been noted after driving the sand drains with a closed mandrel. As the piezometers had been placed after driving the sand drains the reason for this high excess hydrostatic pressure was uncertain. At Napa River the piezometers in the center of the area of four sand drains were placed before the sand drains were driven. In Figure No. 13, the piezometer readings before and after driving sand drains are shown. The excess hydrostatic pressures before driving the sand drains were about equal to the load of the working table (data on April 6, 1960). After driving the sand drains the excess hydrostatic pressure was about 45 percent in excess of the loading due to the working table (data on May 22, 1960). The driving of the sand drains thus produced an abnormally high excess hydrostatic pressure. This high excess hydrostatic pressure could result in failures of working tables that had previously been stable. It is not known whether the displacement of the soil during the driving of the sand drains or the use of air pressure during the withdrawal of the mandrel produced this high excess hydrostatic pressure. The change in excess hydrostatic pressure at the locations where data are available is shown in Table No. 3.

At three locations in the sand drain area piezometers were placed one, two and three feet from the outer edge of a sand drain after the sand drains were driven. A piezometer was also placed in the sand drain. These piezometers were to gain an understanding of how drainage was occurring into the sand drain. One set of these data is shown in Figure No. 14, and is shown as excess hydrostatic pressure variation with time. The pressures in the native soil were constant with distance from the sand drain. The excess hydrostatic pressures as a function of the distance from the sand drain at any given time are shown in Figure No. 15. The data indicate that the expected hyperbolic pressure distribution did not develop, even after a period of several years. This condition existed at all three test locations. The blockage of drainage at the peripheral smear zone appears to be the reason for this condition. As a routine operation, the permeability test was performed on each piezometer after it was installed[9]. A review of these data indicated that the permeability indicated by the piezometers one foot from the sand drain was 1/5 to 1/10 the permeability two to three feet from the sand drain. It would thus appear that the smear zone extends outward from the sand drain one to two feet. The permeability two to three feet from the sand drain was slightly lower than the permeabilities indicated by other piezometers prior to the driving of the sand drains. No data was obtained at the same piezometer location before and after driving sand drains. However, the data is fairly consistent at the three test locations.

Strength of the Foundation Soil

An important consideration in the design of sand drain installations is the increase in strength of the foundation soil under load. Where sand drains are used to increase the stability of the fill, the increase in strength expected will determine the rate of loading that could be utilized. To measure this increase in strength of the foundation soil an extensive boring program was used at the Napa River test section.

The sample borings were made with a 2-inch inside diameter fixed piston type samplers. A thin wall extension point was used to reduce disturbance. The sampler was forced into the foundation soil by means of a hydraulic pressure device on a drill rig. When sampling was performed through the fill, an 8-inch auger was used to make a hole through the fill. The strength of the foundation soil was then determined by means of the unconfined compression test. At various times the vane borer was used to measure the in situ shearing strength of the foundation soil. The vane borer used was constructed by the Materials and Research Department of the California Division of Highways, and is mdeled after the Swedish vane borer(4). The shear strength as determined by the unconfined compression test and the vane borer were in reasonable agreement.

The soft foundation soil at the west approach to the Napa River showed an increase in shearing strength with depth. The strength at the surface approached zero pounds per square foot and increased about ten pounds per square foot for each foot of depth. At about elevation -50 to -60 layers of sandy silt and peat existed. The strength became erratic at this depth. Underlying this layer was a stiff silty clay layer with a shearing strength of 600 to 1,000 pounds per square foot. This stiff silty clay layer extended to a depth of about 200 feet and has numerous sand layers in it.

To determine the effect of the driving of the sand drains upon the strength of the foundation soil three borings were made one day after the sand drains in three different areas were driven. A typical set of data is shown in Figure No. 16. The borings that were made after the sand drains were driven were an equal distance from four sand drains. The data indicates that no significant change in shearing strength of the foundation soil occurred when the sand drains were driven.

Two areas were designated as strength test sections in the nonsand drain areas where borings were to be made at various times to determine the strength of the foundation soil. Areas were chosen where sample borings prior to construction were available, where uniform fill conditions were planned, and where settlement and excess hydrostatic pressure data were available. One was on centerline under 15 feet of fill and one in the berm left of centerline. The data obtained from the area on centerline are shown in Figure No. 17. This area is near the settlement indicated in Figure No. 7. There was a small increase in shearing strength indicated by the boring data at six months after completion of the fill. The boring data indicates that the strength of the foundation soil has continued to increase slowly. The data indicates a 50 percent increase in strength has occurred to this time in 1965. This is a relatively small increase in shearing strength for a 4-year period. The other strength test area in the nonsand drain area indicated that about the same strength increase occurred at that location.

Three areas were designated as strength test sections in the sand drain areas where borings were to be made at various times to determine the strength of the foundation soil. These areas were chosen as in the nonsand drain areas.

A set of representative data at one of the test locations is shown in Figure No. 18. This test area is near the settlements shown in Figure No. 8. All borings in the sand drain area were made equal distance from four sand drains. At the time of failure on the left side there was about a 25 percent increase in shearing strength of the foundation soil in the sand drain area. About two years after the completion of the fill the shearing strength of the foundation soils had increased by 50 to 100 percent, increasing about 200 pounds per square foot at all depths. Four years after completion of the fill the strength of the foundation soil had increased to 600 to 1,000 pounds per square foot which is about what the ultimate shearing strength was estimated to be under the loading of 24 feet of fill.

The shearing strength of the soft foundation soil did not increase as rapidly as was estimated prior to construction. It was estimated that the upper 20 feet of the foundation soil would increase about 50 to 75 pounds per square foot in the nonsand drain area during the loading of the fill. It was estimated that the foundation soil in the sand drain area would increase in shearing strength about 100 pounds per square foot during loading. Neither of these strength increases occurred. This lack of strength increase resulted in the failure of the fill.

Moisture Decrease

One measure of the rate of consolidation of a foundation soil is the decrease in moisture content. The moisture content is also related to the strength of a given soil. Two methods of following the moisture change in the soft silty clay at the Napa River experimental sand drain section were used. The moisture content of the soil was measured from the samples obtained from the borings. The other method of measuring moisture changes was by the use of nuclear subsurface gages.

The moisture contents at Napa River decreased with depth. The moisture content was about 110 percent of the dry weight at elevation -10. The moisture decreased lineally with depth to a moisture of about 75 percent at elevation -50. The moisture contents before construction are shown in Figures Nos. 17 and 18.

The moisture content of the soft silty clay in the nonsand drain areas showed a minor decrease four years after construction. See Figure No. 17. Two years after completion of the fill the moisture decrease was about 3 percent and after four years was about 10 percent.

The moisture content of the soft foundation soil as determined from the samples from borings in the sand drain areas did not show a decrease during construction. See Figure No. 18. Two years after construction the moisture content had decreased 15 percent and after four years had decreased 20 to 25 percent.

Assuming that all of the measured settlement is due to drainage of water from the soil, the decreases in moisture content are within reasonable agreement with the measured settlements.

This is true in both the sand drain and nonsand drain areas.

The nuclear gages measure the moisture content of the soil in pounds of water per cubic foot. The moisture content of the soil is therefore reported in these units in this report. These units of pounds of water per cubic foot are a volume measure of moisture content. The nuclear access tubes failed by corrosion after 1962 so that no data is available after this date.

The moisture content as determined by the nuclear gages in the nonsand drain area is shown in Figure No. 19. The moisture content in the nonsand drain area did not show a significant decrease in this two-year period. This is in agreement with the moistures determined from the samples from the borings.

The moisture content as measured by the nuclear gages in the sand drain area is shown in Figure No. 19. No significant change in moisture content is indicated during construction, which is in agreement with the moistures determine from the samples from the borings. During the first two years after construction there was a one and one-half to two pounds of water per cubic foot decrease in moisture content indicated in the sand drain area. This is in reasonable agreement with the moisture change noted by the sample borings.

Movement Measurements

In order to better understand the way in which the failure occurred, two methods of measuring movements were used. Heave stakes were used to measure the movement of the surface of the fills. Both horizontal and vertical movements were measured.

The movement of the foundation soil was measured by means of a slope indicator.

The heave stake lines were installed on the berms. The failure to the right side produced a heave of the existing roadway and are shown in Figure No. 3. The rate of movement increased at the time of failure, and gave some notice that movement was occurring.

The reloading of the fill was resumed on September 8, 1960, and the rate of movement on the left side continued at the same rate as prior to reloading. From September 21 to 27 a slight increase in the rate of movement of the heave stakes occurred. See Figure No. 4. However, this movement was not considered alarming. The readings on September 30th, after the failure had occurred indicated a large horizontal movement had occurred.

The main deficiency in the use of heave stakes is the time required to perform the necessary surveying, reduction and analysis of the data. A time lag of one day was normal. Also large manpower would be required to make the necessary measurements once or twice daily. For this reason heave stakes are limited in the routine application to stability control during normal construction.

The slope indicators were installed when the working table was completed. Two slope indicators were installed ten to fifteen feet from the edge of the future sand drain area. The lateral movement of the foundation soil caused by the driving of the sand drains is shown in Figure No. 20. The soft foundation soil was moved laterally during the contractor's operations of placing the sand drains. This may be due to the use of a closed mandrel.

The data obtained from a slope indicator to the left of the fill in the area that failed is shown in Figure No. 21. This installation was destroyed during construction and reinstalled after the berms were widened on the left. As loading of the main fill was resumed, after the failure on the right, a minor movement was noted at a depth of 25 feet below the top of the tubing. On the morning of September 27th this installation was read. When the data was reduced and plotted on the norming of the 28th, after the failure, it was found that the slope indicator had indicated a major movement occurred at a depth of 25 feet. This indicates how the time delay is critical in stability problems. On September 28, 1960, it was found that the tubing had failed at a depth of about 20 feet.

A series of long-term readings are shown in Figure No. 22 from an installation to the right side. During the loading of the main fill prior to September 27, 1960, a minor movement occurred at a depth of 15 feet. This movement has continued to 1965 at a fairly uniform rate and is now at the limit of the slope indicator scale. This indicates a movement of the foundation soil is occurring which has not been recorded by the heave stakes. This is evidently a plastic type flow that is slowly occurring. This condition has been noted on several other slope indicators that have long-time readings. However, no cracking or mud waves have been observed.

The heave stakes and slope indicators are both useful installations. However, they both have one deficiency for use as a stability control device during routine construction. This Defifiency is the time required to obtain the necessary results.

Conclusion

This experimental sand drain project produced several items of information of benefit to the design and construction of future sand drain projects.

1. The consolidation of the soft foundation soil in sand drain areas with similar conditions as existed at Napa River will occur at a rate two to two and one-half times slower than estimated from theoretical studies.
2. Sufficient time must be available to construct fills on soft soils at a slow rate. It may be necessary to construct fills on soft soils under separate contracts and utilize stage construction.
3. Where sand drains are installed in soft foundation soils similar to the conditions at Napa River, it should be accomplished by a method that will not disturb the soft foundation soil.
4. With the slow rate of consolidation of the soft foundation soils, sand drains may be required under struts as well as under the main fill.

Acknowledgments

The research described in this paper was conducted by the California Division of Highways, Materials and Research Department. The Materials and Research Department was under the supervision of Mr. F. N. Hveem (retired) and Mr. J. L. Beaton. The work was performed by the Foundation Section under the direction of Mr. A. W. Root (retired), and Mr. Travis W. Smith.

The construction of the fill was under the supervision of the California Division of Highways District 10, under Mr. J. G. Meyer, District Engineer. The Construction Engineer was Mr. W. F. Fleharty, and the Resident Engineer Mr. L. E. Daniel.

References

1. Barron, Reginald A. "Consolidation of Fine-Grained Soils by Drain Wells," American Society of Civil Engineers Transactions, Vol. 113, 1948, p 718.
2. Cedergren, H. R. and Weber, W. G., "Subsidence of California Highways," American Society for Testing Materials Special Technical Publication No. 322, Field Testing of Soils, 1962.
3. Weber, W. G., "Construction of a Fill by a Mud Displacement Method," Highway Research Board Proceedings, Vol. 41, 1962, p 591.
4. Cadling, Lyman, and Odenstad, Sten, "The Vane Borer," Royal Swedish Geotechnical Institute Proceedings No. 2, Stockholm 1950.
5. Weber, W. G., "Measurement of Excess Hydrostatic Pressures in Soils," American Society for Testing Materials, Special Technical Publication No. 254, 1959.
6. Wilson, Stanley D., "Detection of Landslides," Eleventh Northwest Conference on Road Building, University of Washington, February 21, 1958.
7. Belcher, D. J., Cuykendall, T. R., and Sack, H. S., "The Measurement of Soil Moisture and Density by Neutron and Gamma-Ray Scattering." U.S. Civil Aeronautics Admin. Tech. Development Report No. 127, 1950.
8. Taylor, Donald W., "Fundamentals of Soil Mechanics," John Wiley and Sons.

9. "Time Lag and Soil Permeability in Ground Water Observation," Bulletin No. 36, Waterways Experiment Station, Corps of Engineers, U.S. Army, Vicksburg, Mississippi.

10. Schiffman, Robert L., "Field Applications of Soil Consolidation Under Time-Dependent Loading and Varying Permeability," Highway Research Board Bulletin 248, 1960.

TABLE No. 1

Properties of Soft Bay Mud at Napa River

Properties	Elevation -10 ft.	Elevation -50 ft.
Moisture % Dry Wt.	110	65
Compressive Strength PSF	225	885
Wet Density PCF	85	95

	Percent
Sand	0
Silt	40
Clay	60
Plastic Index	38
Liquid Limit	87

TABLE NO. 2

Grading of Filter Material

Sieve Size	Percent Passing Sand Drain	Percent Passing Permeable Blanket
1"		100
3/4"		97
3/8"	100	68
No. 4	97	52
8	73	40
16	53	29
30	36	20
50	16	11
100	5	6
200	3	5

TABLE NO. 3

EXCESS HYDROSTATIC PRESSURE
BEFORE AND AFTER
DRIVING SAND DRAINS

(Fill Loading 0.33 Tons Per Square Foot)

Piezometer No.	Elevation	Excess Hydrostatic Pressure in Tons per Square Foot Before	After	Increase
45	-4	.29	.38	.09
46	-12	.15	.41	.26
71	-4	.32	.37	.05
72	-16	.29	.59	.30
73	-23	.30	.58	.28
74	-36	.35	.42	.07
83	-4	.29	.36	.07
84	-16	.30	.46	.16
85	-25	.30	.52	.22
86	-36	.30	.67	.37
88	-4	.32	.42	.10
89	-15	.38	.47	.09
90	-25	.32	.54	.22
91	-35	.33	.56	.23
92	-4	.28	.44	.16
95	-16	.28	.46	.16
96	-26	.32	.48	.16
97	-36	.34	.56	.26

PLAN OF TEST SITE

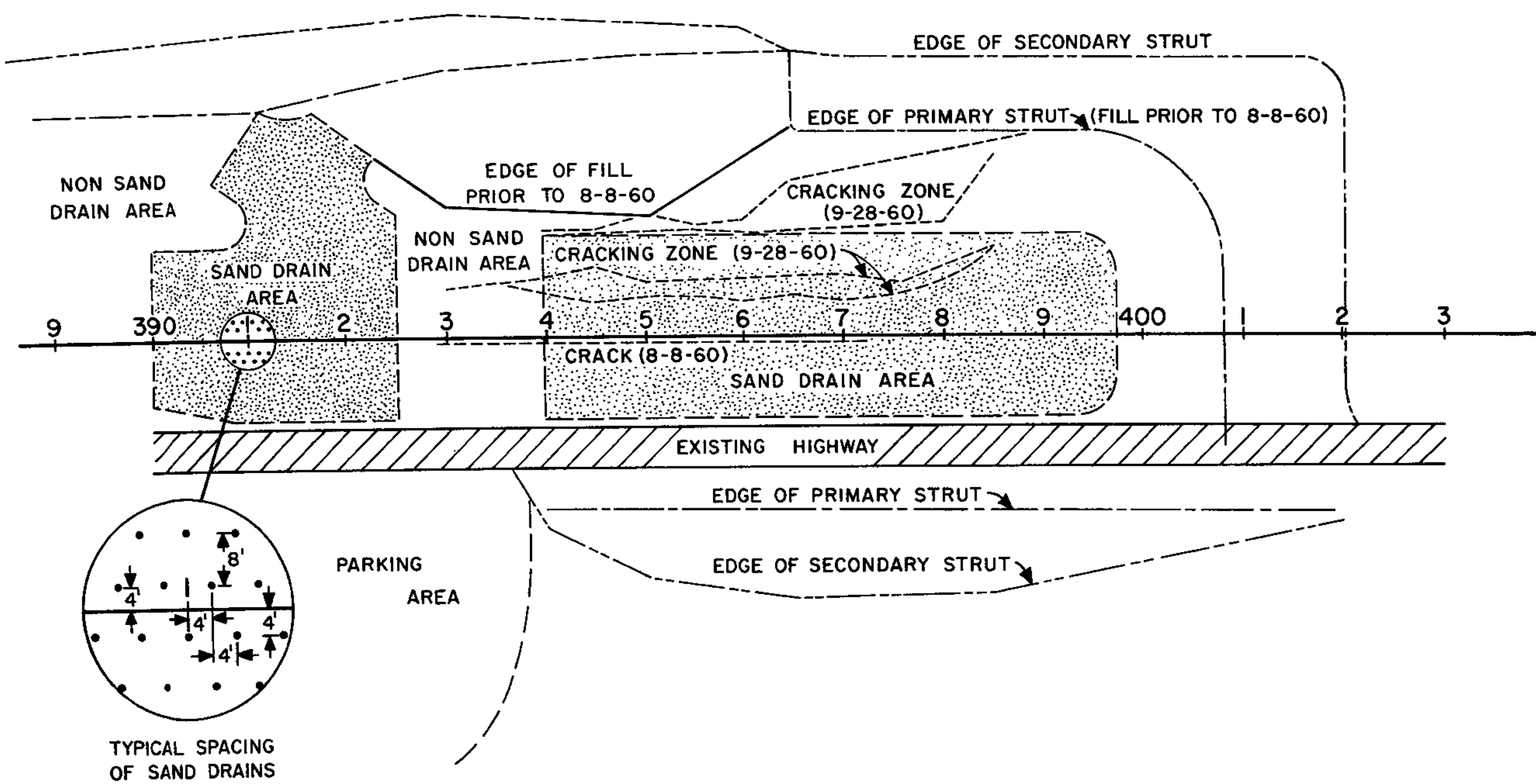
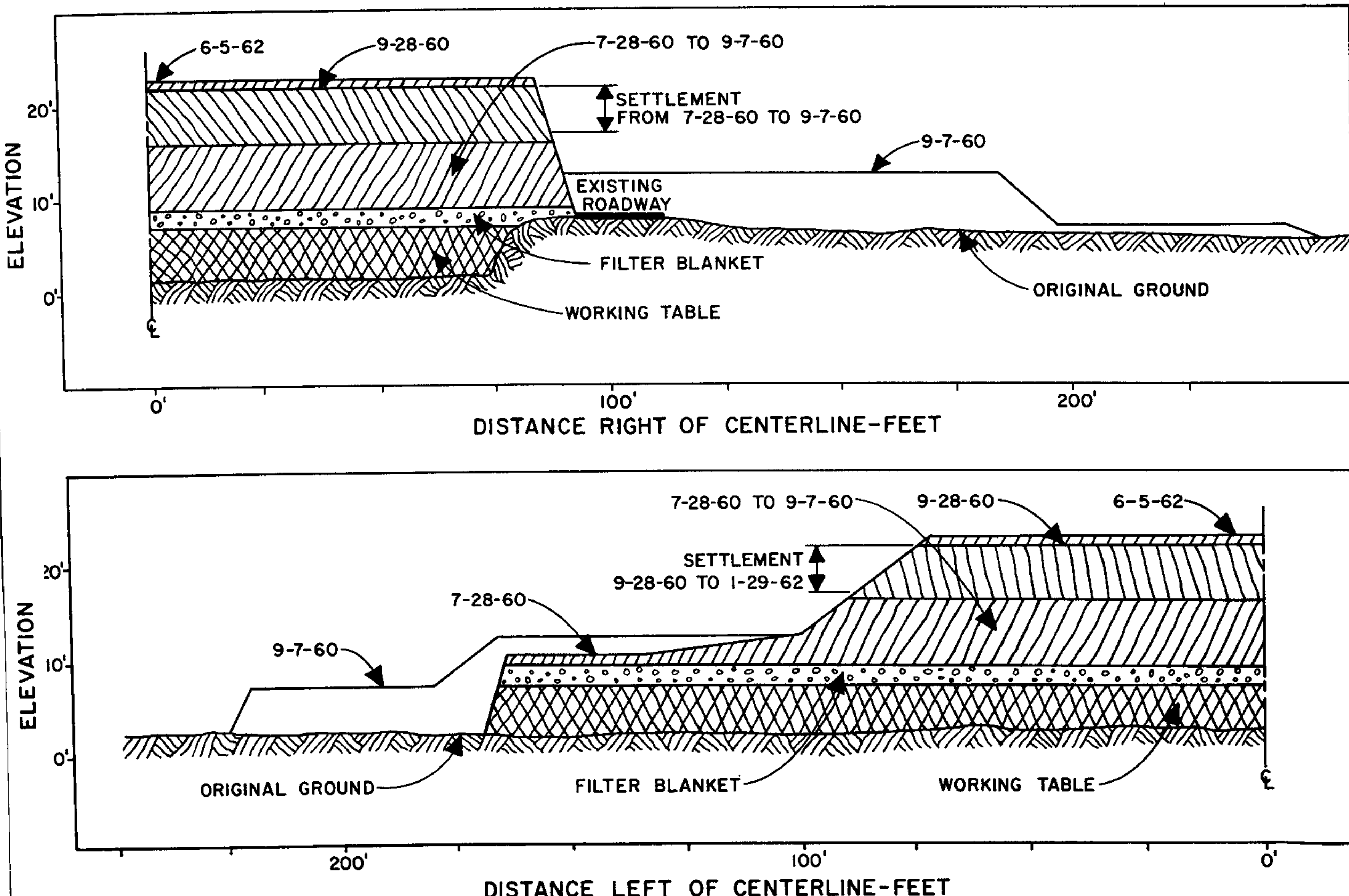


FIGURE 1

TYPICAL CROSSECTIONS



MOVEMENT ON RIGHT SIDE

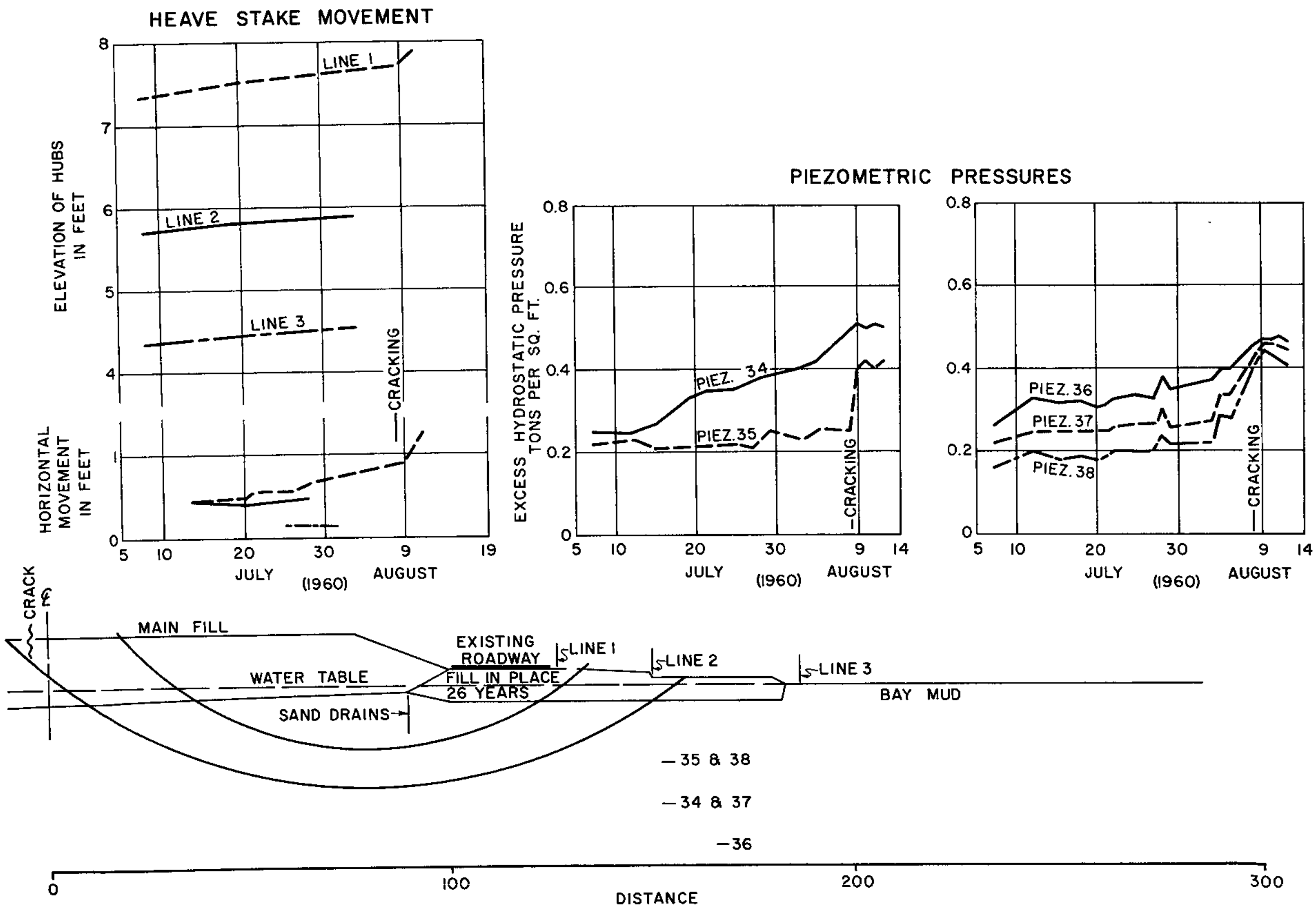


FIGURE 3

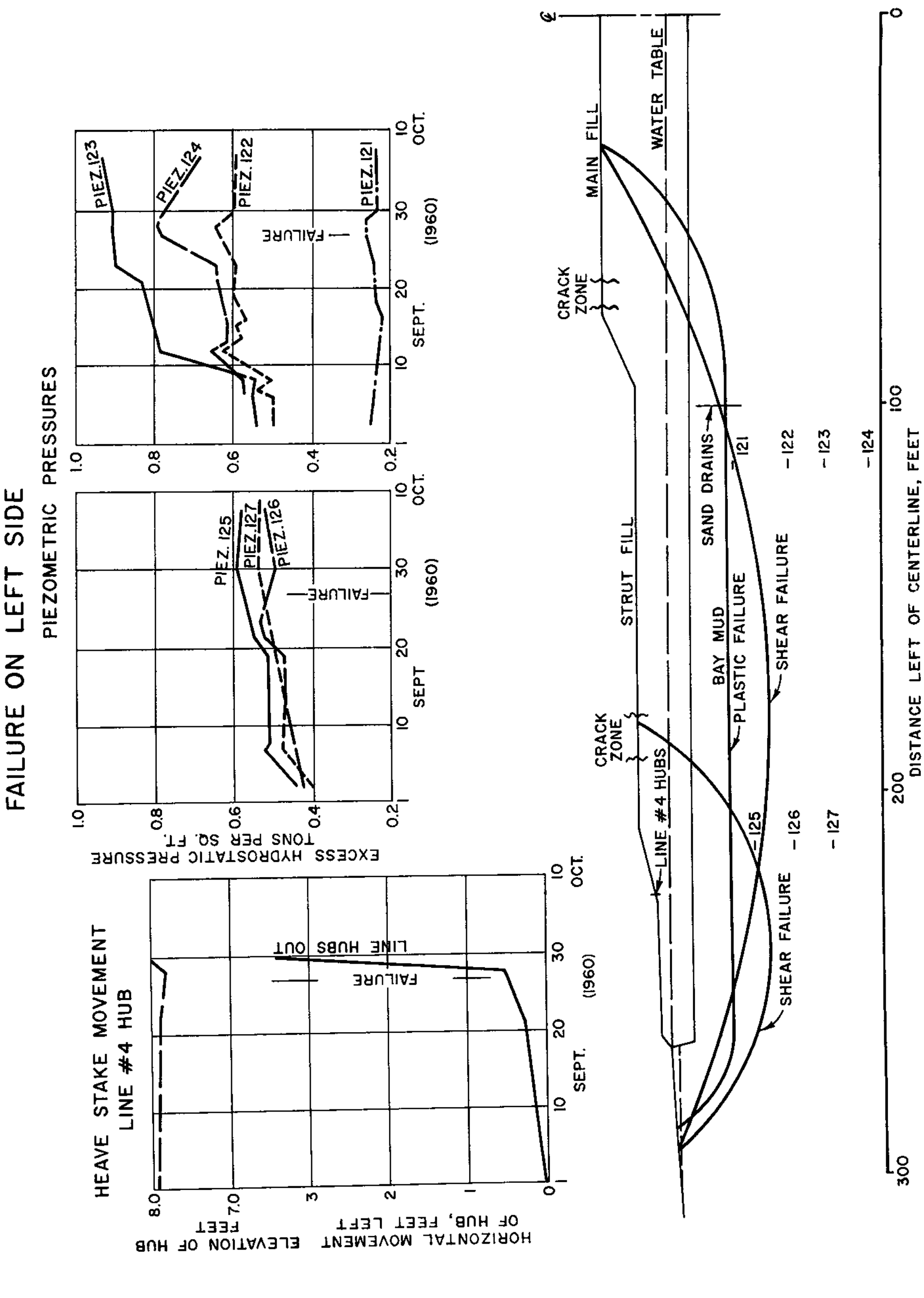

FAILURE ON LEFT SIDE
PIEZOMETRIC PRESSURES
PIEZ.123
PIEZ.124
PIEZ.122
PIEZ.121
PIEZ.125
PIEZ.127
PIEZ.126
FAILURE
SEPT.
OCT.
(1960)
EXCESS HYDROSTATIC PRESSURE
TONS PER SQ. FT.
HEAVE STAKE MOVEMENT
LINE #4 HUB
LINE HUBS OUT
HORIZONTAL MOVEMENT OF HUB, FEET LEFT
ELEVATION OF HUB FEET
MAIN FILL
WATER TABLE
CRACK ZONE
STRUT FILL
SAND DRAINS
BAY MUD
PLASTIC FAILURE
SHEAR FAILURE
LINE #4 HUBS
DISTANCE LEFT OF CENTERLINE, FEET

FIGURE 5

SETTLEMENT PROFILE IN SAND DRAIN AND NON SAND DRAIN AREAS

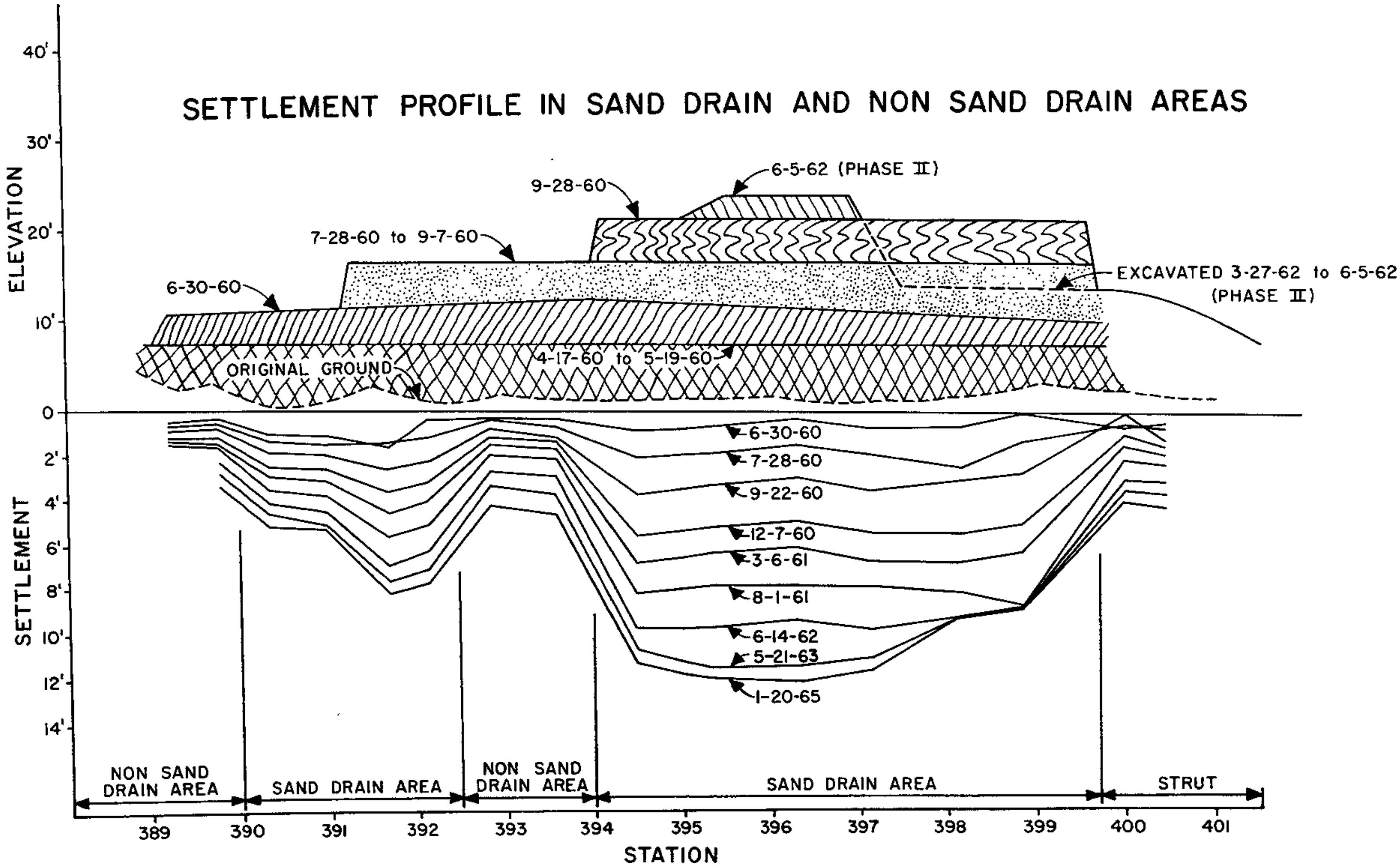


FIGURE 7

COMPARISON OF THEORETICAL AND MEASUREMENT SETTLEMENTS
NAPA RIVER 15 FEET OF FILL

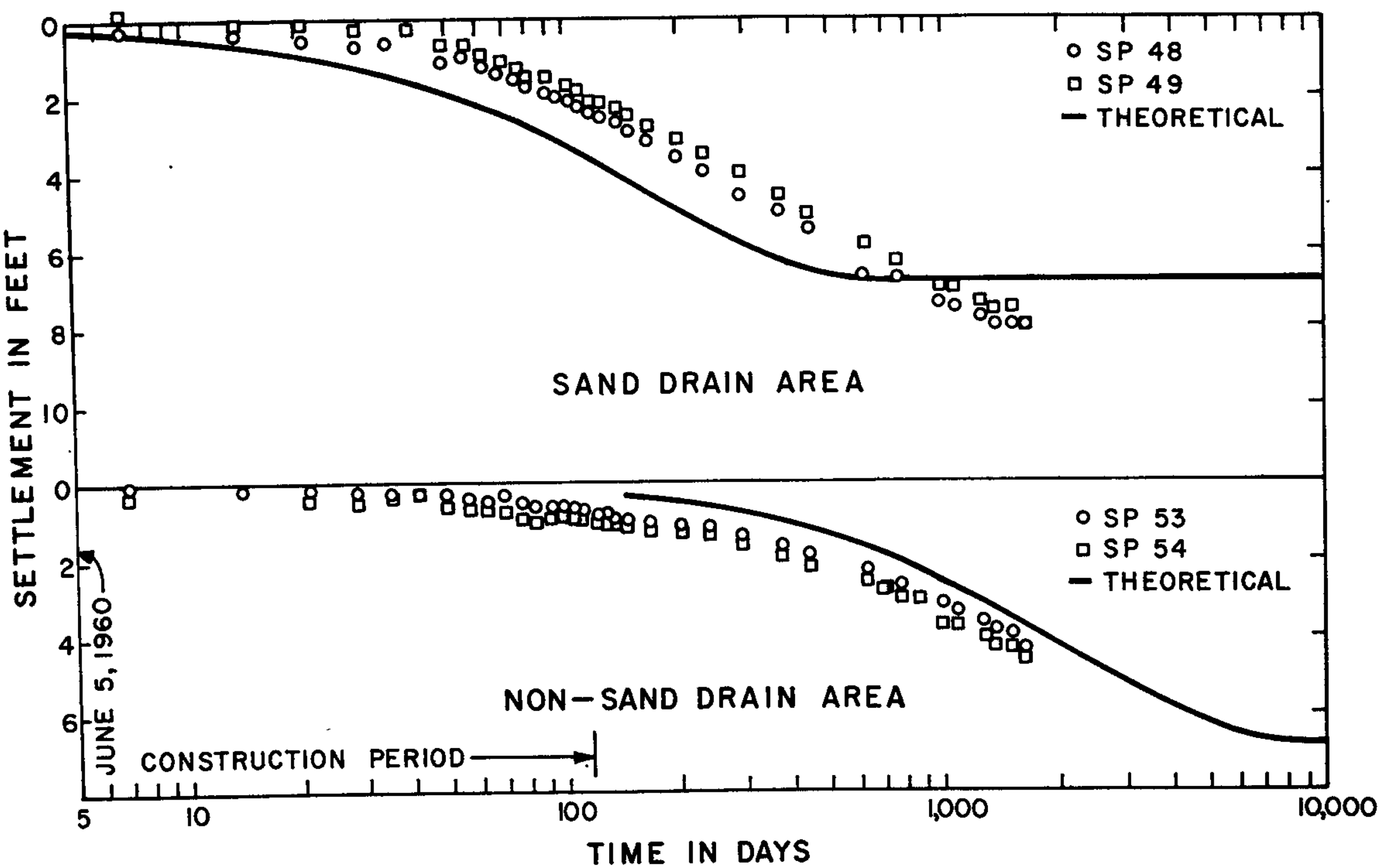
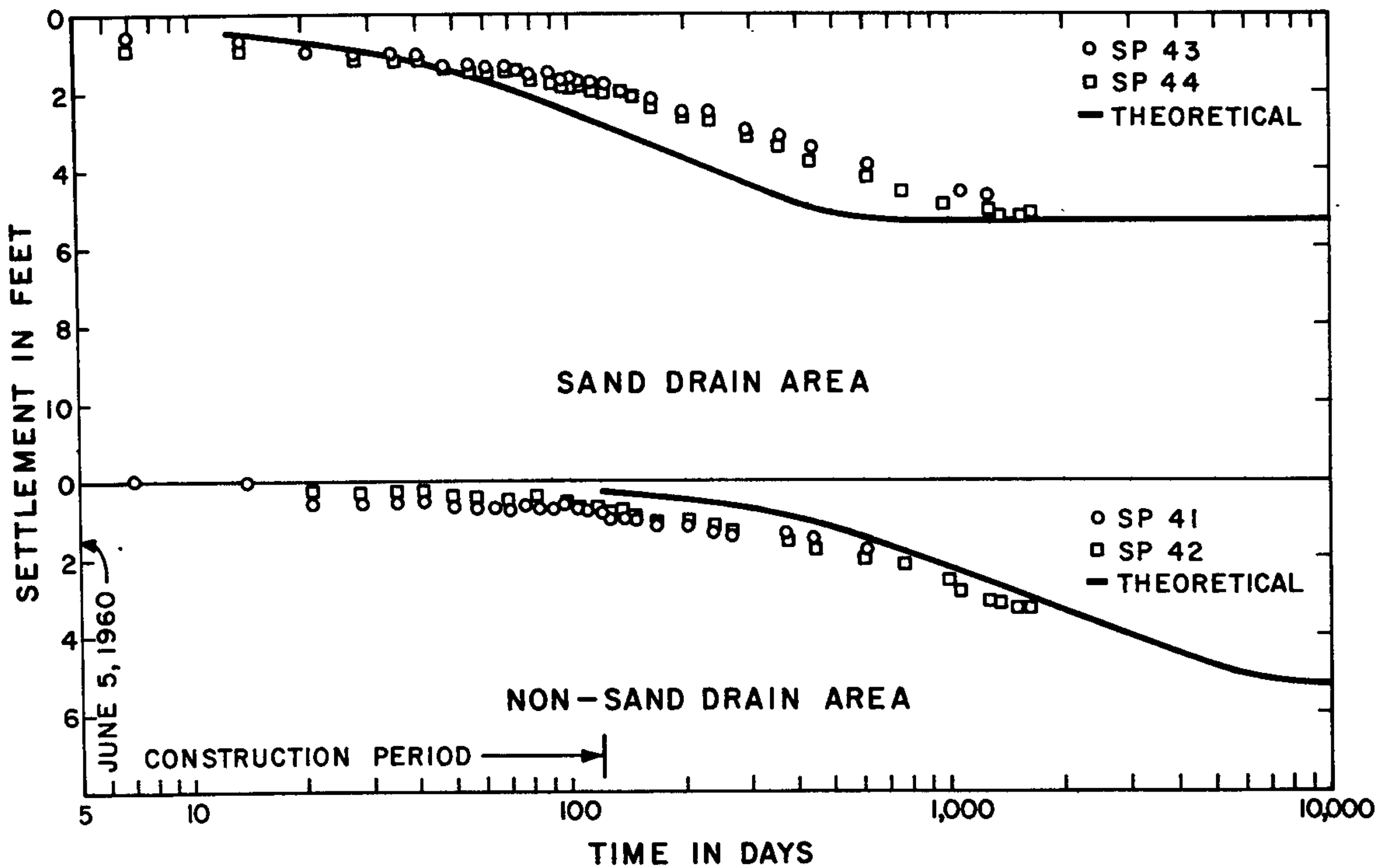
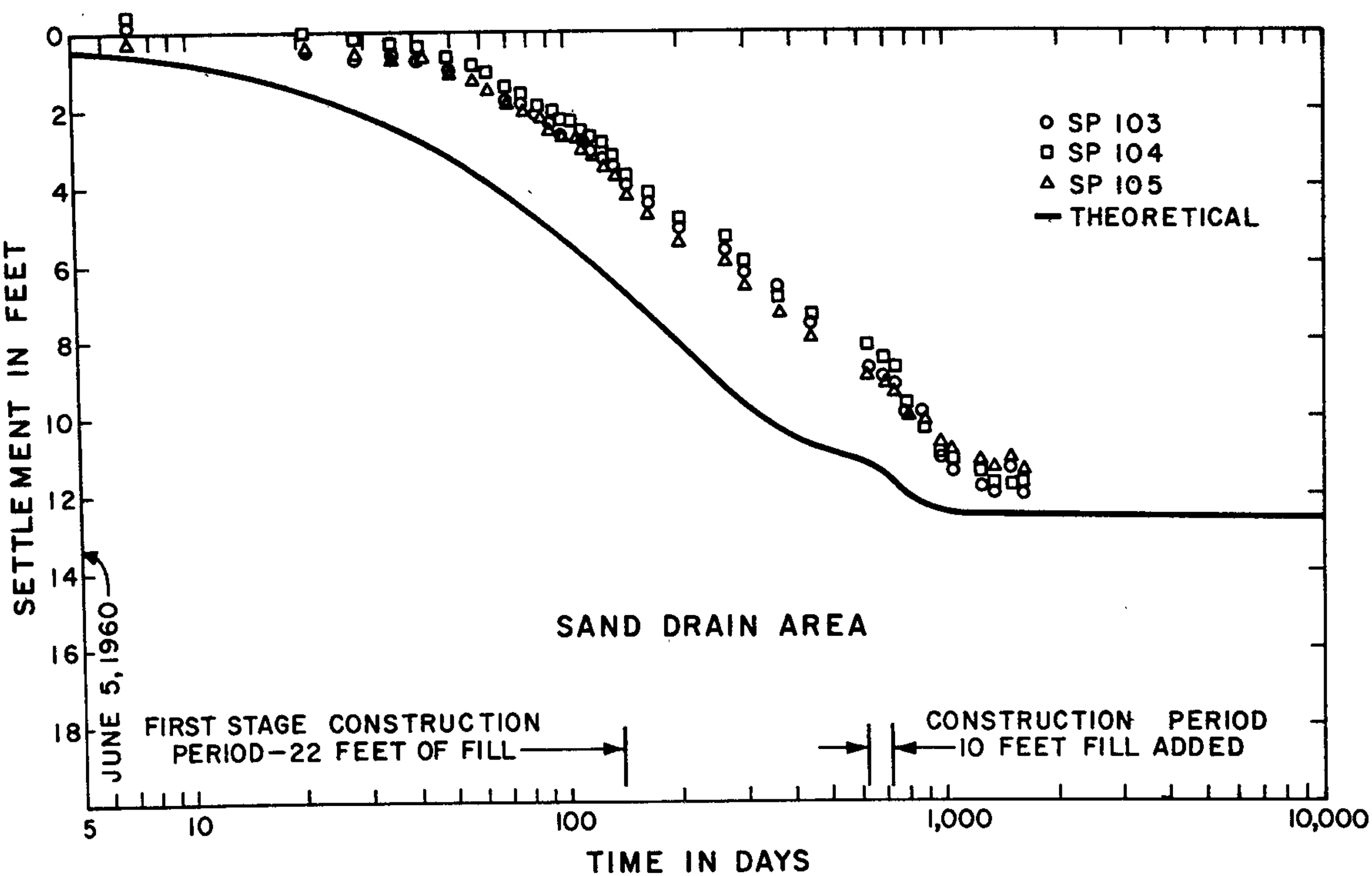


FIGURE 6

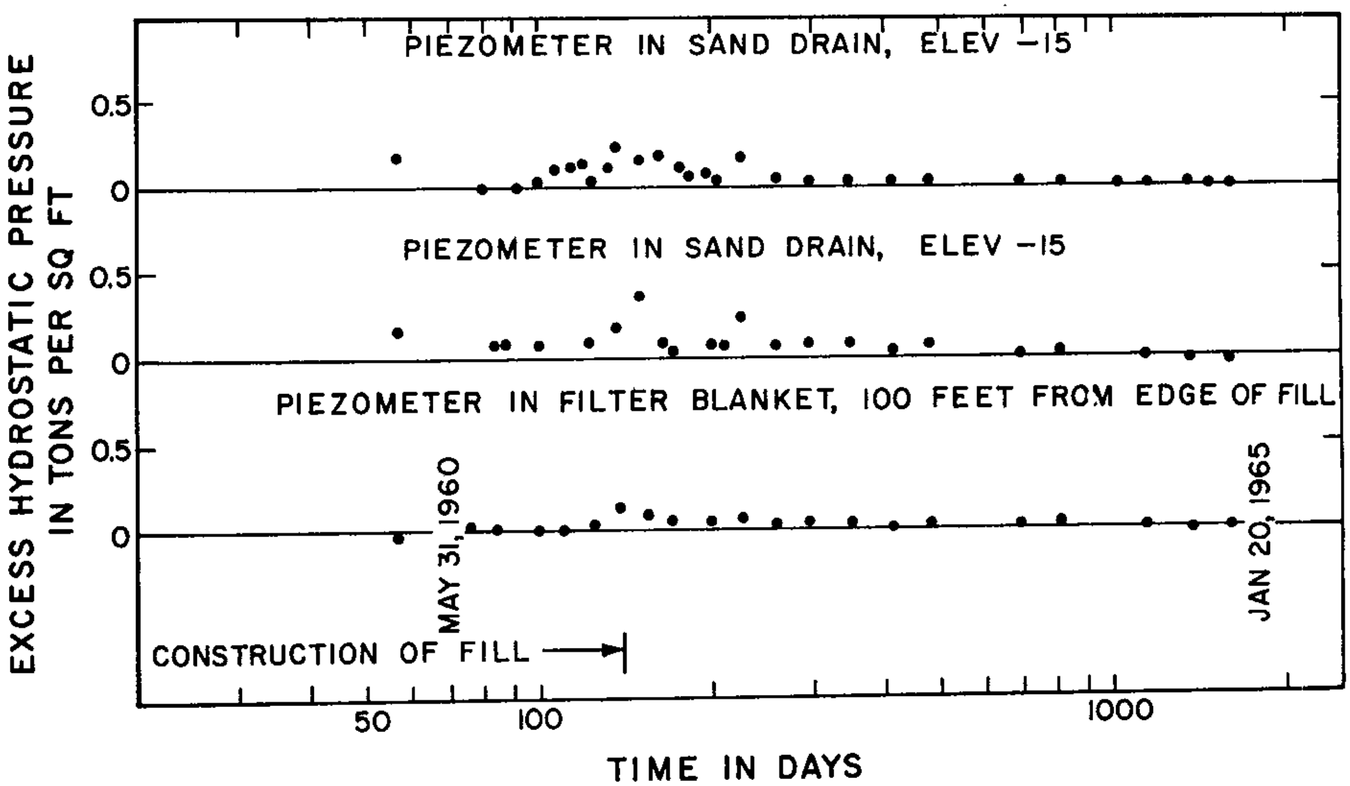
COMPARISON OF THEORETICAL AND MEASUREMENT SETTLEMENTS
NAPA RIVER 10 FEET OF FILL



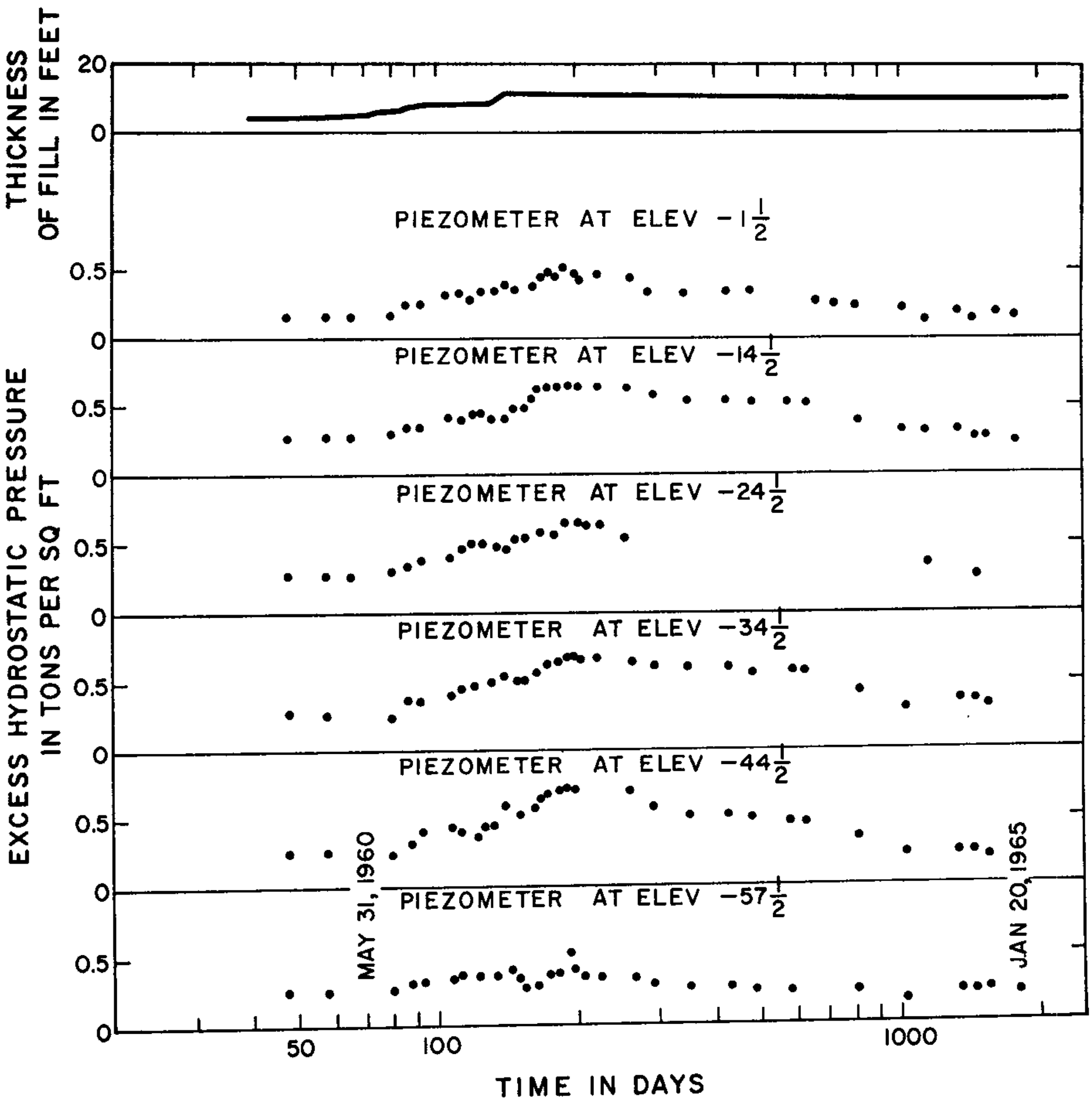
COMPARISON OF THEORETICAL AND MEASUREMENT SETTLEMENTS
NAPA RIVER 32 FEET OF FILL IN TWO STAGES



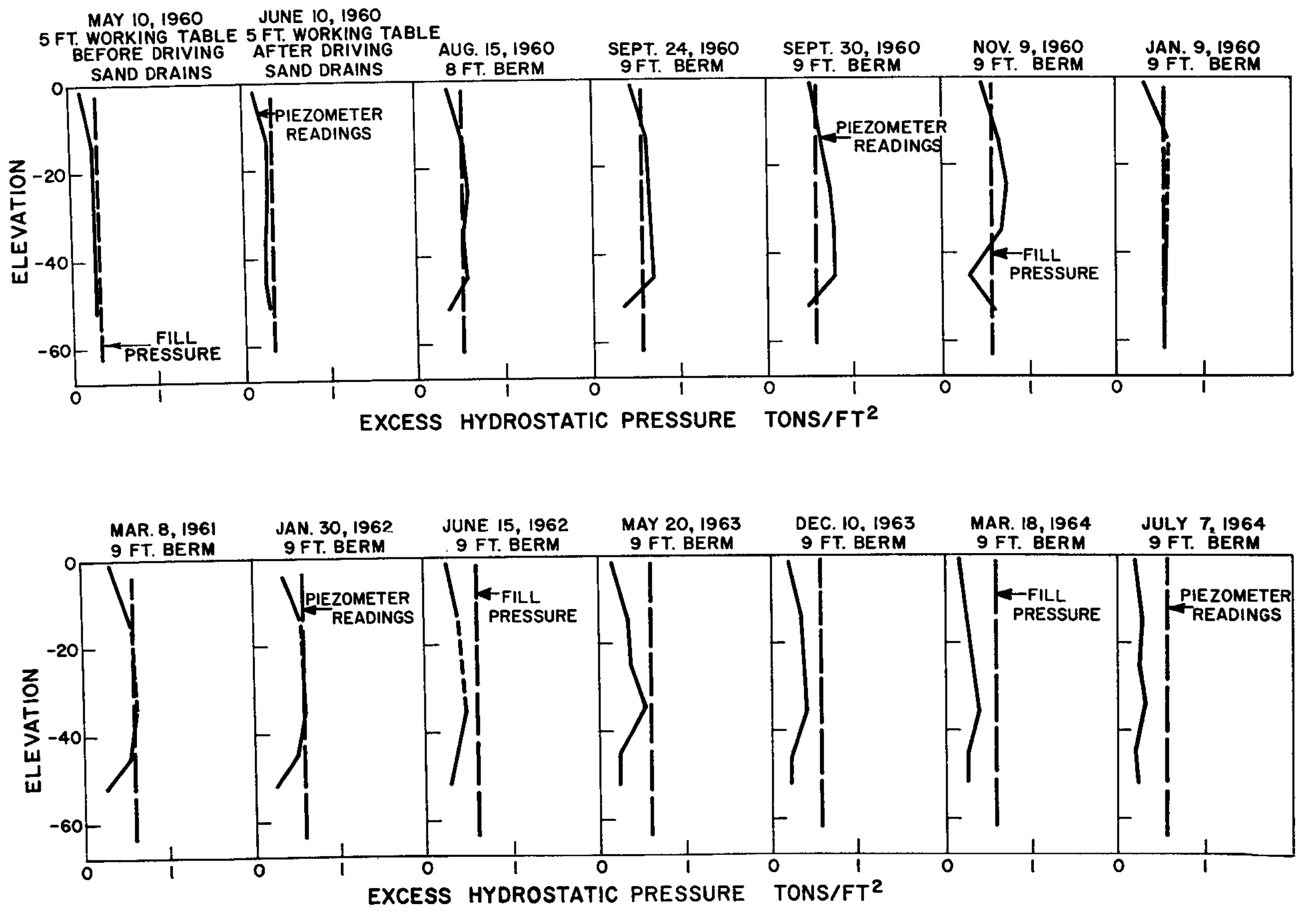
WATER PRESSURE IN FILTER SYSTEM



VARIATION OF EXCESS HYDROSTATIC PRESSURE WITH DEPTH NON SAND DRAIN AREA



VARIATION OF EXCESS HYDROSTATIC PRESSURE WITH DEPTH FOR VARIOUS STAGES OF CONSTRUCTION
NON SAND DRAIN AREA



VARIATION OF EXCESS HYDROSTATIC PRESSURE WITH DEPTH
SAND DRAIN AREA

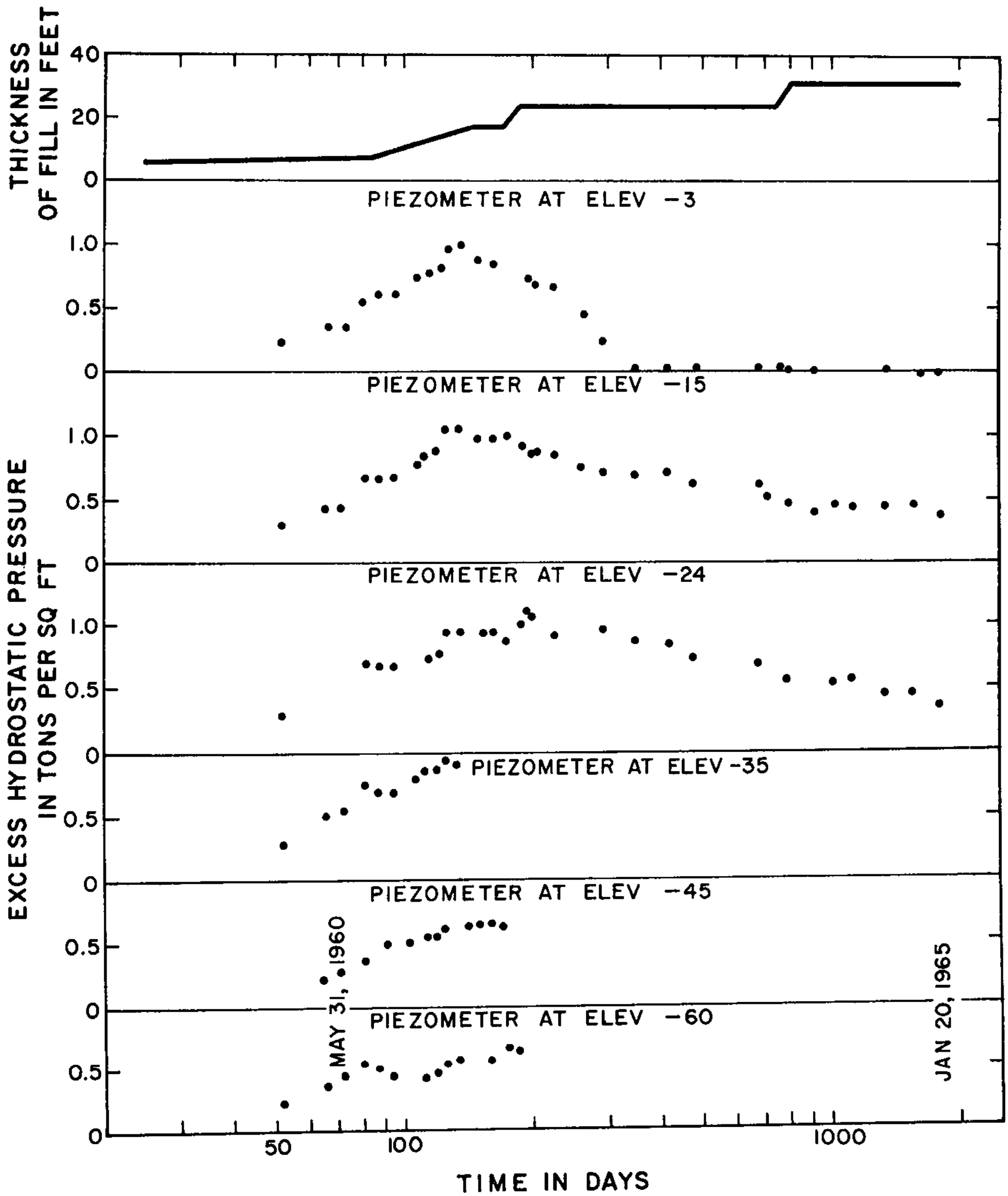
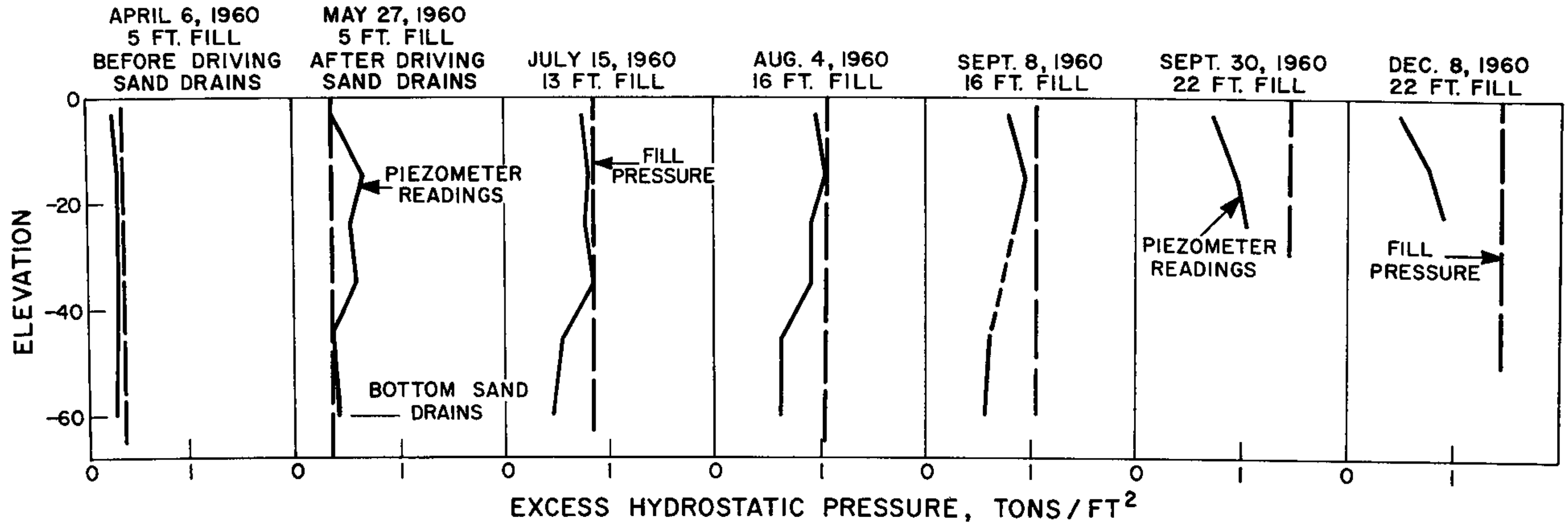
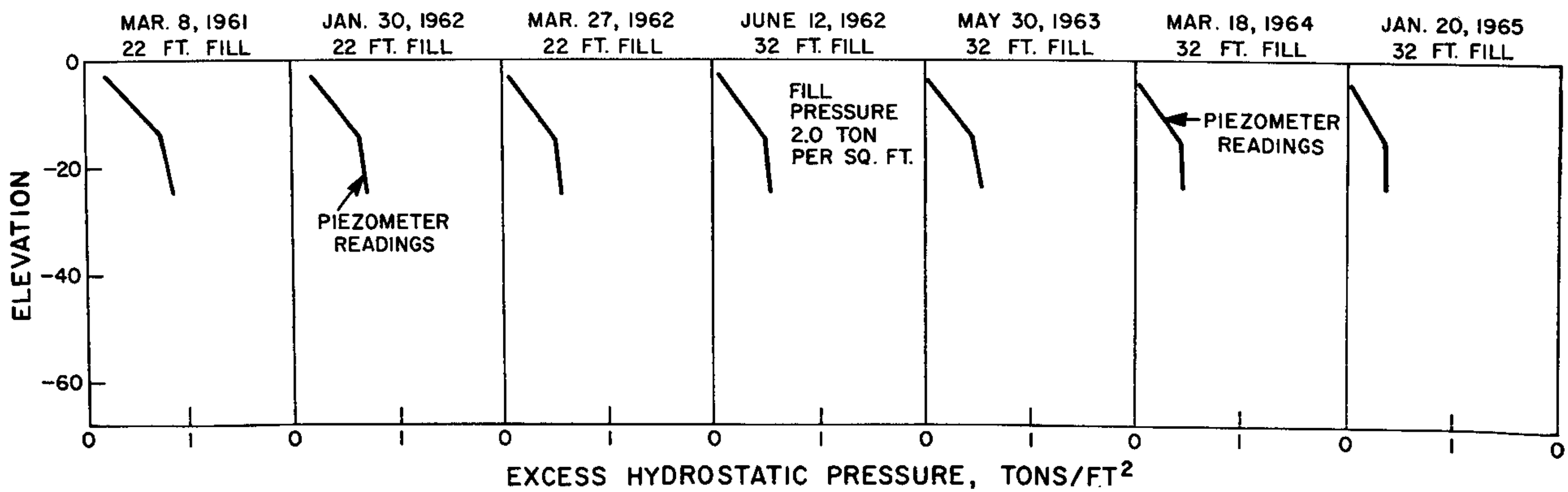


FIGURE 13

VARIATION OF EXCESS HYDROSTATIC PRESSURE WITH DEPTH FOR VARIOUS STAGES OF CONSTRUCTION

SAND DRAIN AREA





VARIATION OF EXCESS HYDROSTATIC PRESSURE WITH DISTANCE FROM SAND DRAIN

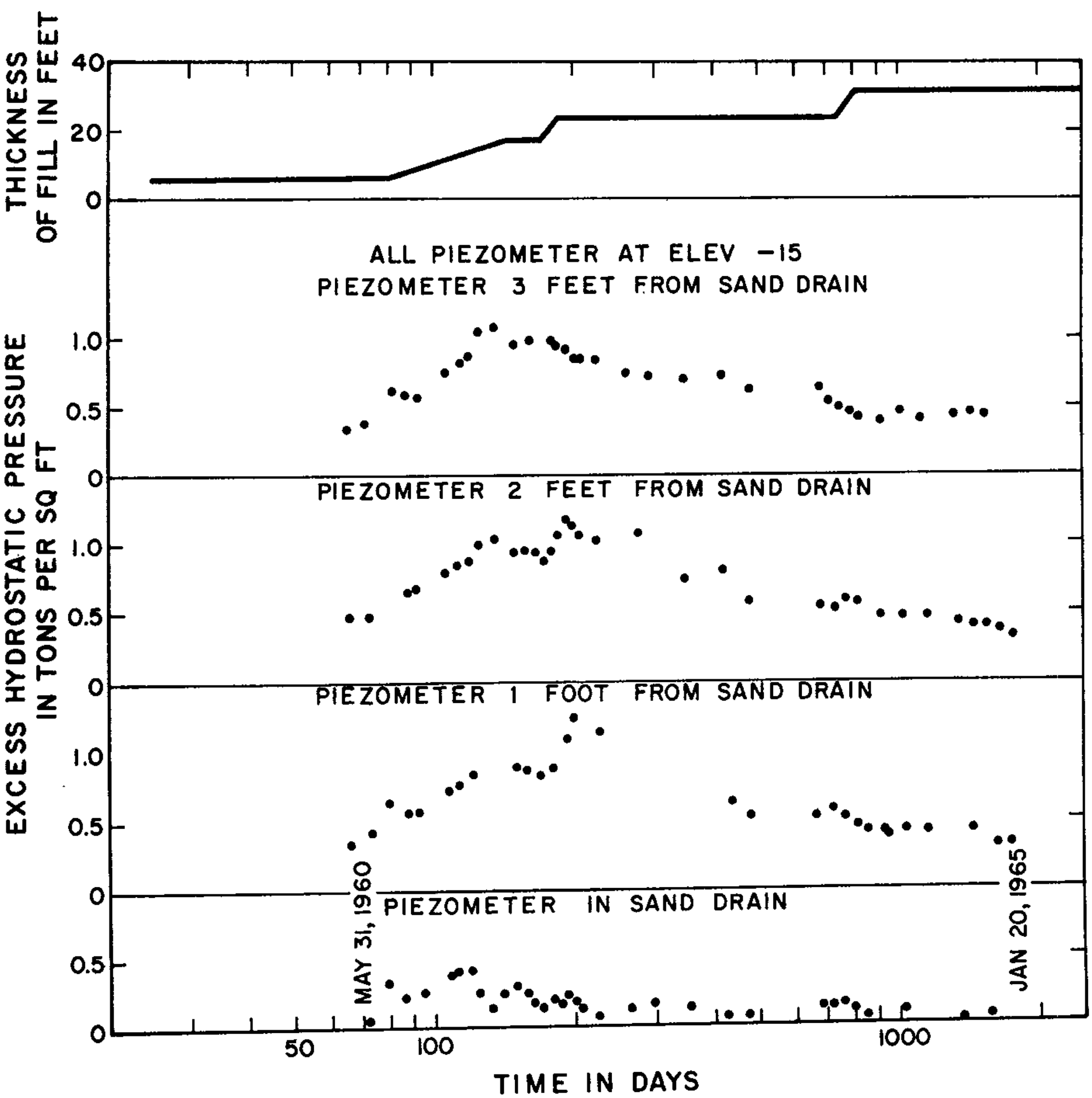
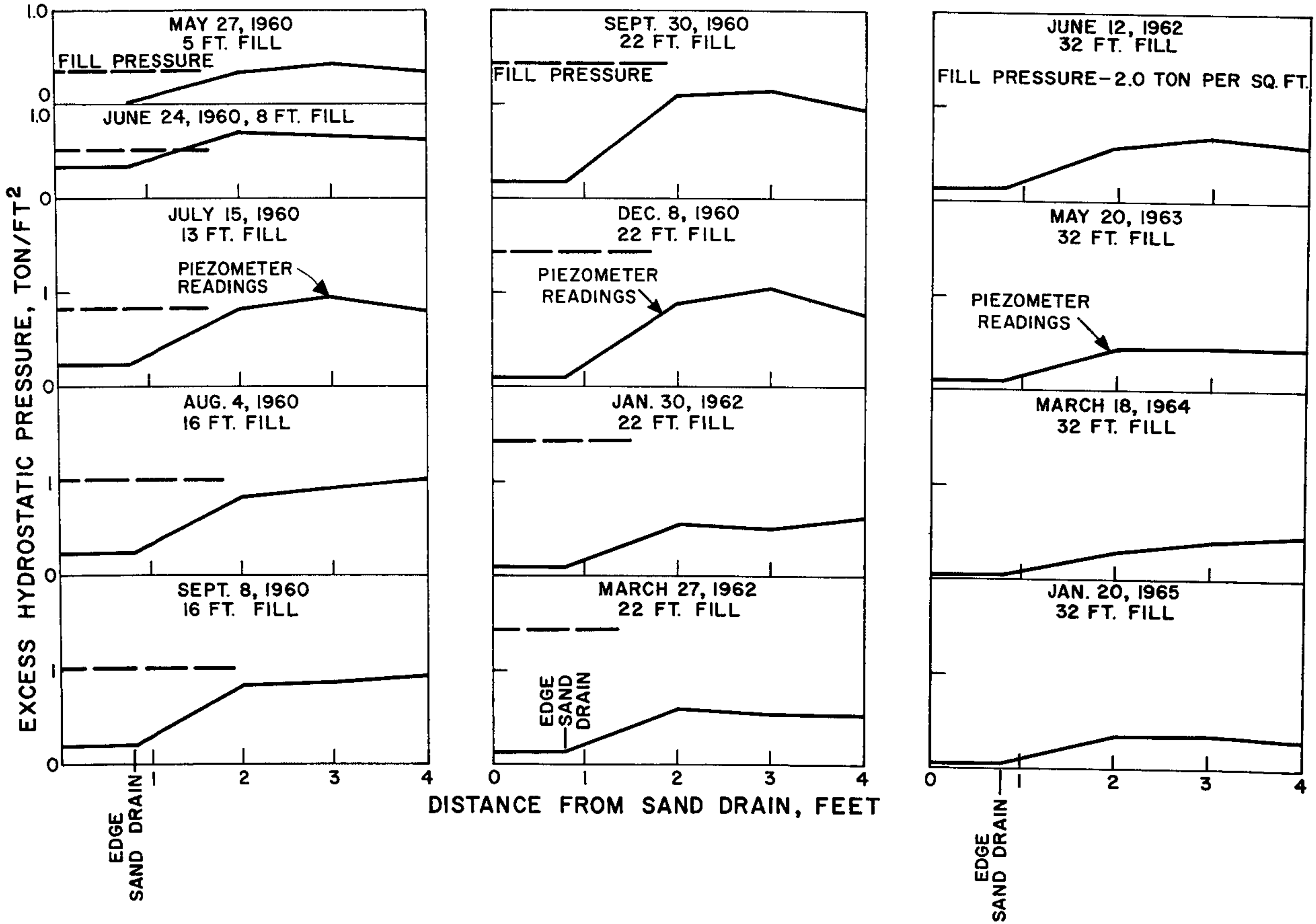


FIGURE 15

VARIATION OF EXCESS HYDROSTATIC PRESSURE WITH DISTANCE FROM SAND DRAIN



COMPARISON OF SHEARING STRENGTH WITH DEPTH

SAND DRAINS ON EIGHT FOOT SPACING

BORINGS D-21 & 98
PRIOR TO CONSTRUCTION

BORING 164
ONE DAY AFTER INSTALLING
SAND DRAINS

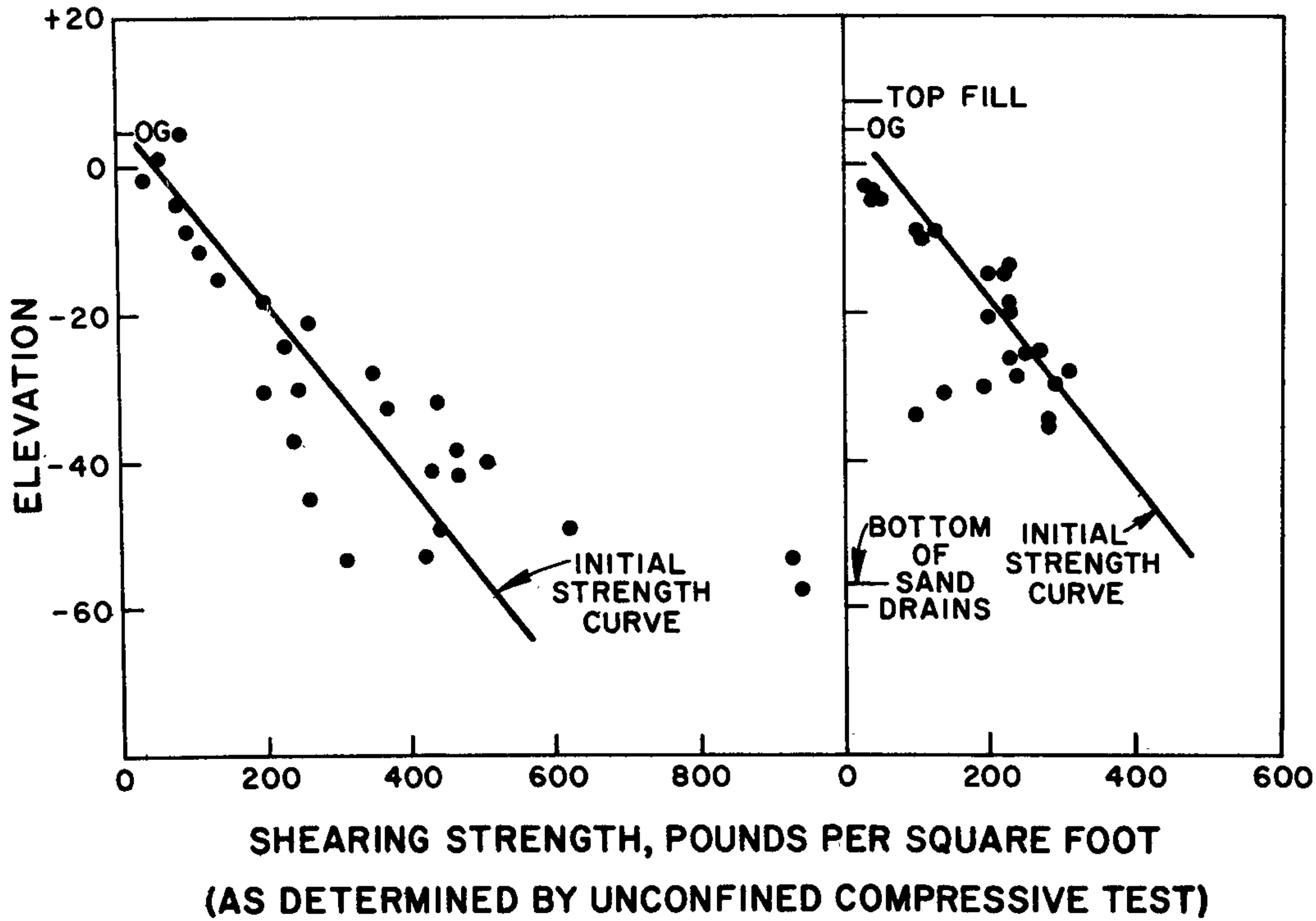


FIGURE 16

COMPARISON OF SHEARING STRENGTH AND MOISTURE WITH DEPTH

NON SAND DRAIN AREA

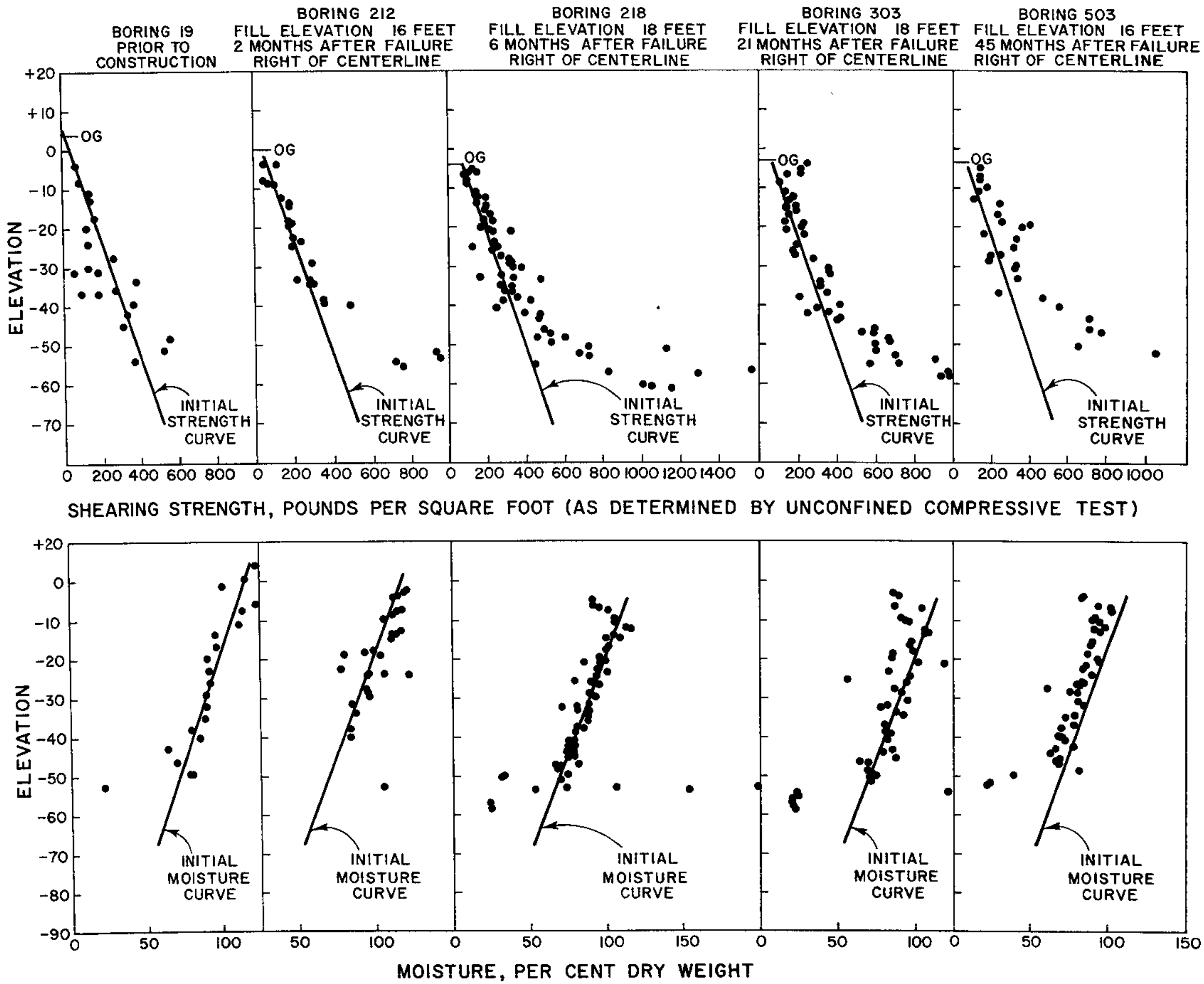
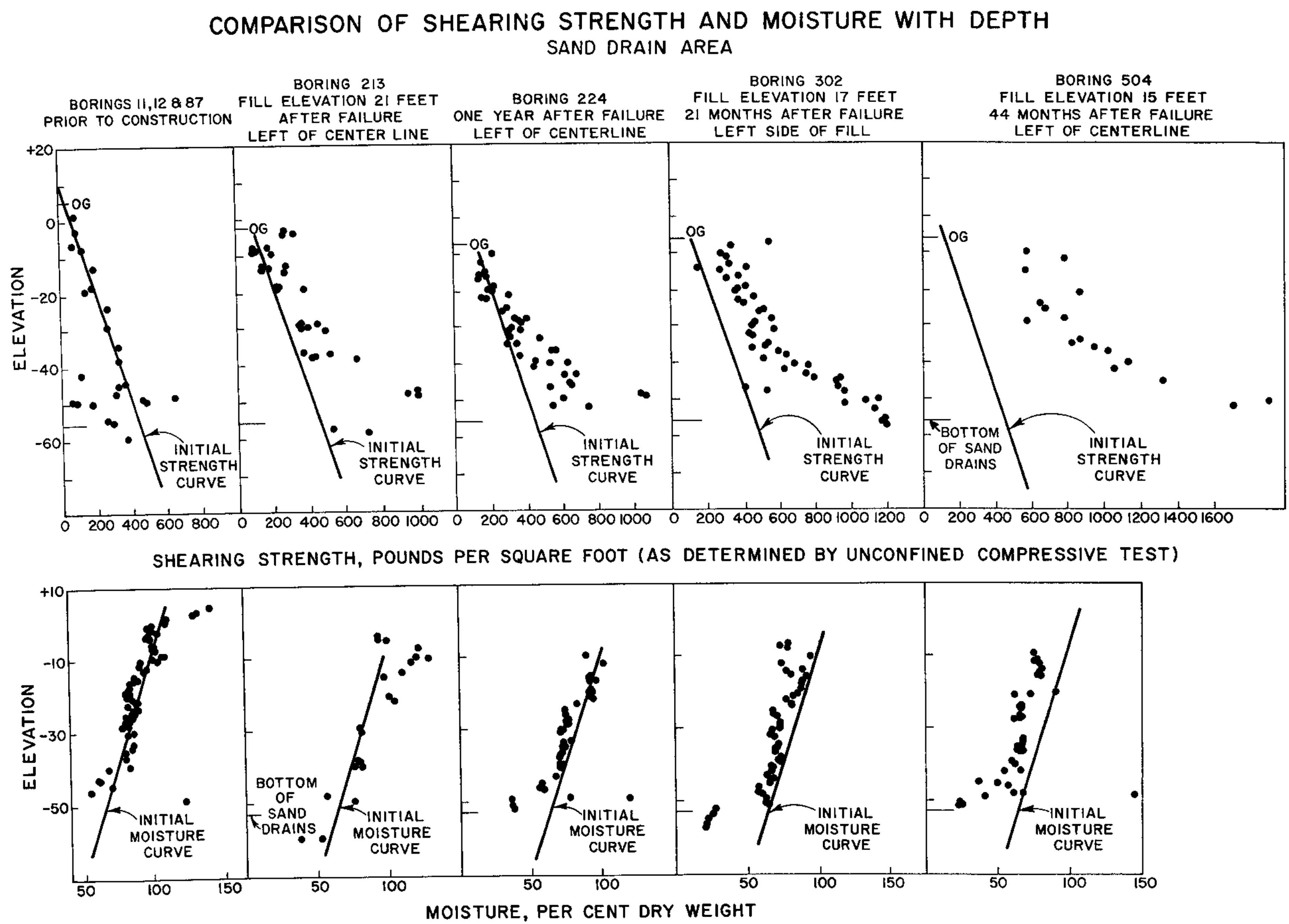
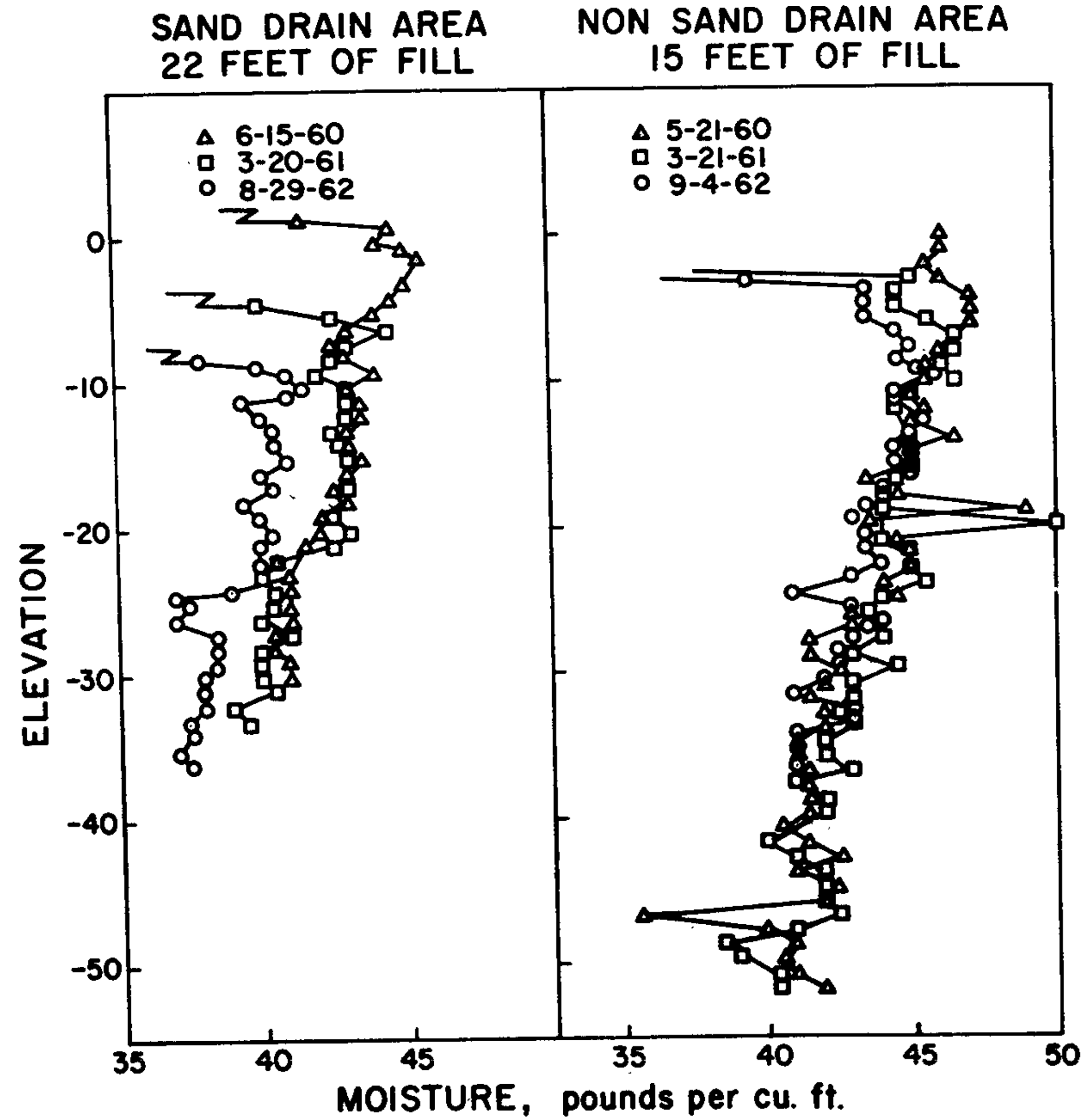


FIGURE 18

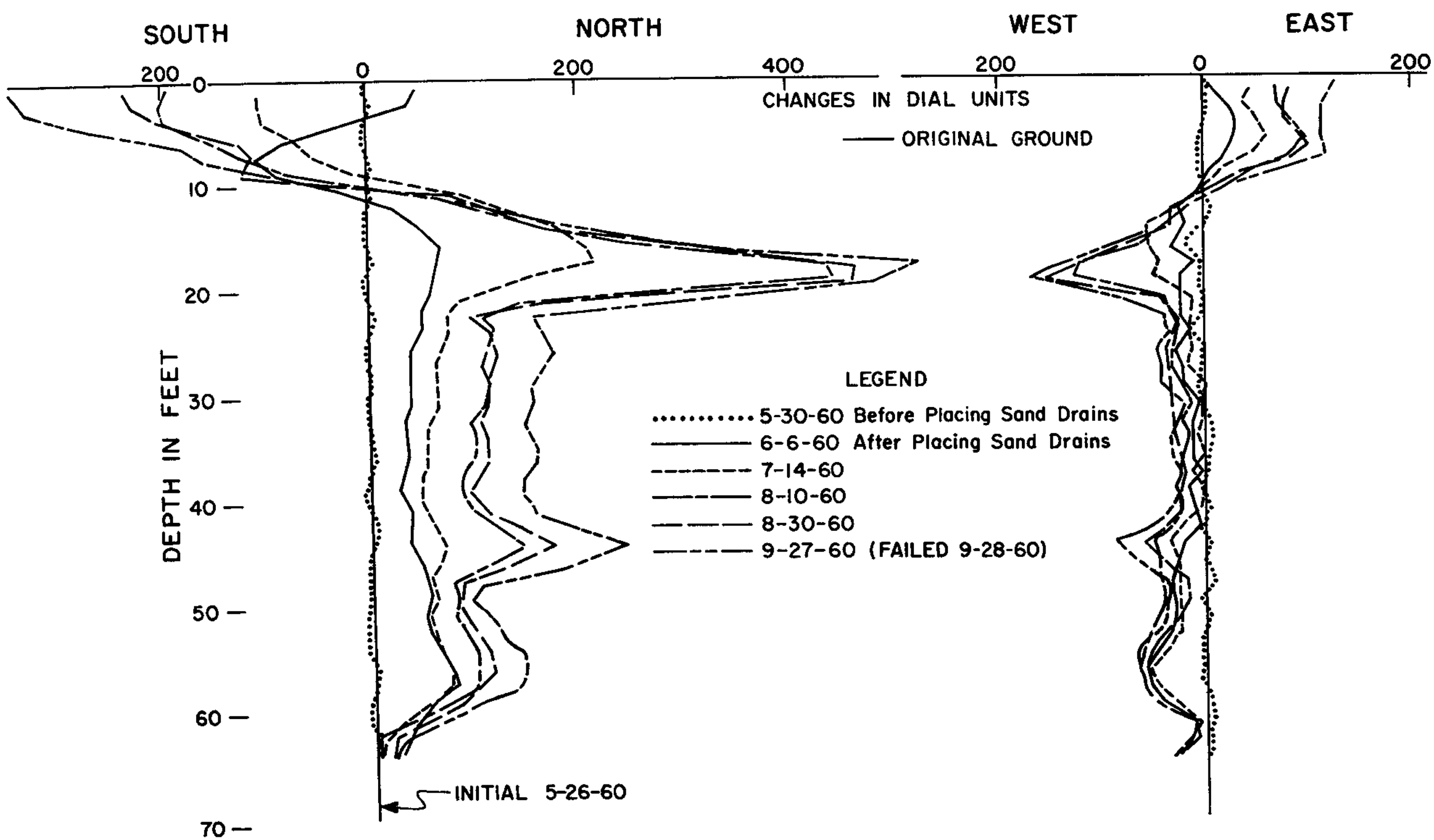


COMPARISON OF SHEARING STRENGTH AND MOISTURE WITH DEPTH
SAND DRAIN AREA
BORINGS 11,12 & 87
PRIOR TO CONSTRUCTION
BORING 213
FILL ELEVATION 21 FEET
AFTER FAILURE
LEFT OF CENTER LINE
BORING 224
ONE YEAR AFTER FAILURE
LEFT OF CENTERLINE
BORING 302
FILL ELEVATION 17 FEET
21 MONTHS AFTER FAILURE
LEFT SIDE OF FILL
BORING 504
FILL ELEVATION 15 FEET
44 MONTHS AFTER FAILURE
LEFT OF CENTERLINE
OG
INITIAL STRENGTH CURVE
BOTTOM OF SAND DRAINS
ELEVATION
SHEARING STRENGTH, POUNDS PER SQUARE FOOT (AS DETERMINED BY UNCONFINED COMPRESSIVE TEST)
INITIAL MOISTURE CURVE
MOISTURE, PER CENT DRY WEIGHT

MOISTURE VARIATION WITH DEPTH AS DETERMINED BY SUBSURFACE NUCLEAR GAGES



MOVEMENT MEASURED BY SLOPE INDICATOR



MOVEMENT MEASURED BY SLOPE INDICATOR

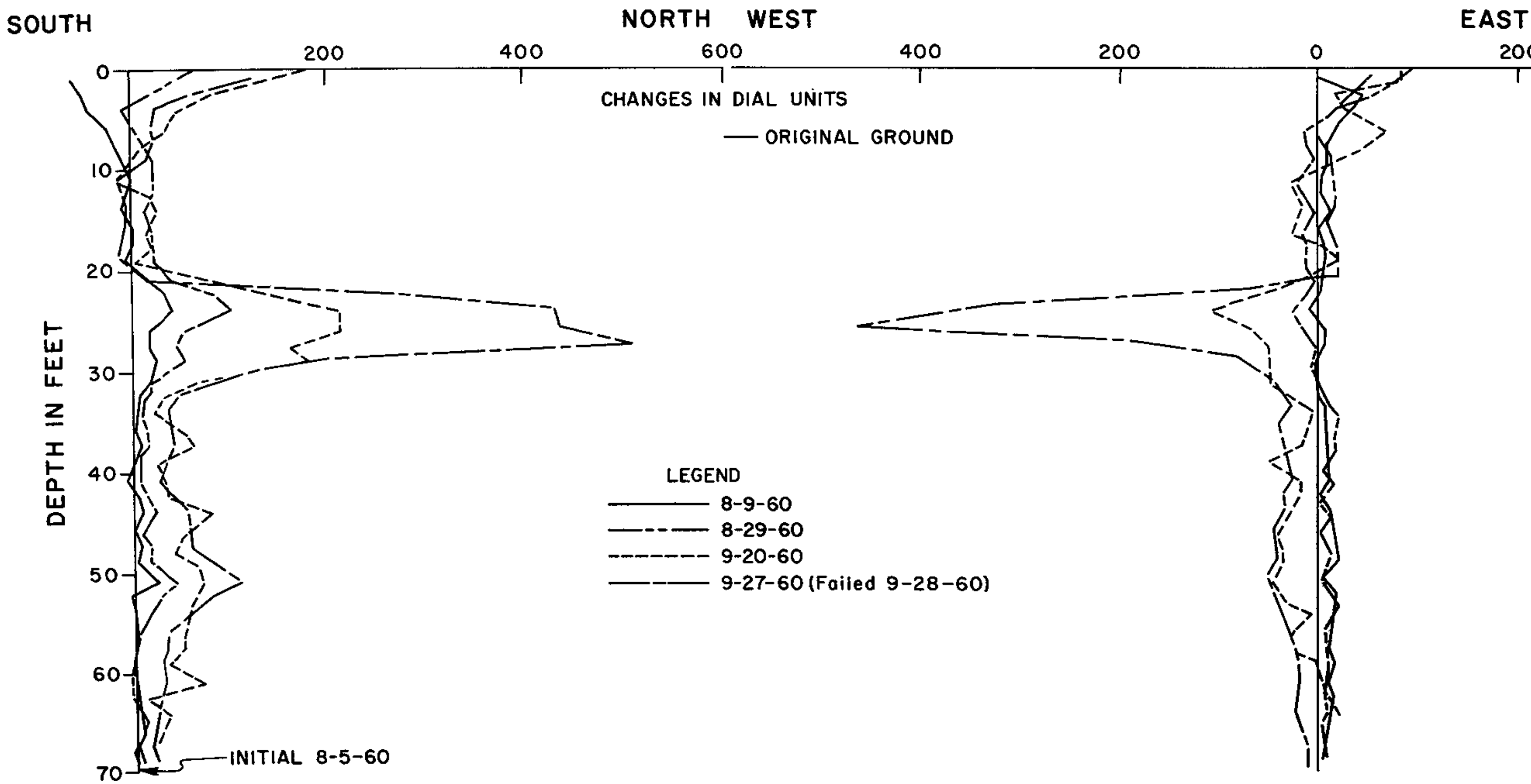


FIGURE 21

FIGURE 22

MOVEMENT MEASURED BY SLOPE INDICATOR